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DREDGING OPERATIONS TECHNICAL
SUPPORT PROGRAM

TECHNICAL REPORT D-90-2

METHODOLOGY FOR ANALYSIS OF SUBAQUEOUS SEDIMENT MOUNDS

by

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US ARMY ENGINEER WATERWAYS EXPERIMENT STATION

February 1990

Final Report

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

ENVIRONMENTAL LABORATORY

The D-series of reports includes publications of the
Environmental Effects of Dredging Programs:

Dredging Operations Technical Support

Long-Term Effects of Dredging Operations

Interagency Field Verification of Methodologies for
Evaluating Dredged Material Disposal Alternatives
(Field Verification Program)

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

1a. REPORT SECURITY CLASSIFICATION
Unclassified

1b. RESTRICTIVE MARKINGS

2a. SECURITY CLASSIFICATION AUTHORITY

2b. DECLASSIFICATION / DOWNGRADING SCHEDULE

3. DISTRIBUTION / AVAILABILITY OF REPORT
Approved for public release; distribution unlimited.

4. PERFORMING ORGANIZATION REPORT NUMBER(S)
Technical Report D-90-2

5. MONITORING ORGANIZATION REPORT NUMBER(S)

6a. NAME OF PERFORMING ORGANIZATION
USAEWES
Environmental Laboratory

6b. OFFICE SYMBOL (If applicable)

7a. NAME OF MONITORING ORGANIZATION

6c. ADDRESS (City, State, and ZIP Code)
3909 Halls Ferry Road
Vicksburg, MS 39180-6199

7b. ADDRESS (City, State, and ZIP Code)

8a. NAME OF FUNDING / SPONSORING ORGANIZATION
US Army Corps of Engineers

8b. OFFICE SYMBOL (If applicable)

9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER

8c. ADDRESS (City, State, and ZIP Code)
Washington, DC 20314-1000

10. SOURCE OF FUNDING NUMBERS

PROGRAM ELEMENT NO.	PROJECT NO.	TASK NO.	WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)
Methodology for Analysis of Subaqueous Sediment Mounds

12. PERSONAL AUTHOR(S)
Poindexter-Rollings, Marian E.

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM ______ TO ______	February 1990	122

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

17. COSATI CODES

FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
Consolidation
Disposal
Dredged materials
Mounds
Site capacity
Soft soil
Subaqueous

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

Dredging of contaminated sediments and subsequent disposal and capping in legally designated disposal sites is an internationally accepted disposal alternative when adherence to strict disposal practices is maintained. As more highly contaminated sediments in the heavily industrialized harbors of the world must be dredged to maintain navigation and economic viability, pressure to use subaqueous dredged material disposal sites will increase. Use of these subaqueous sites has necessitated development of procedures to analyze disposal site capacity based upon physical, chemical, and biological considerations.

A methodology of analysis was developed in this study to investigate the behavior of the created subaqueous sediment mounds. Emphasis was placed upon the physical aspects of mound behavior, although the methodology also includes chemical and biological aspects.

(Continued)

20. DISTRIBUTION / AVAILABILITY OF ABSTRACT
☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS

21. ABSTRACT SECURITY CLASSIFICATION
Unclassified

22a. NAME OF RESPONSIBLE INDIVIDUAL

22b. TELEPHONE (Include Area Code)

22c. OFFICE SYMBOL

DD Form 1473, JUN 86 Previous editions are obsolete.

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

19. ABSTRACT (Continued).

The physical aspects of the methodology were applied to four field sites at which dredged material mounds have been created. The procedure successfully predicted the physical behavior of the constructed dredged material mounds. This method of analysis provides a useful tool for evaluation of subaqueous disposal sites and the dredged material mounds created within these sites; it is equally applicable to analysis of contaminated and uncontaminated dredged material mounds.

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SECURITY CLASSIFICATION OF THIS PAGE

PREFACE

This report was prepared in the Environmental Laboratory (EL), US Army Engineer Waterways Experiment Station (WES), as a part of the Dredging Operations Technical Support (DOTS) Program task area. The DOTS Program is sponsored by the Headquarters, US Army Corps of Engineers (HQUSACE), and is managed by the Environmental Effects of Dredging Programs (EEDP) in the EL. Technical Monitor for the HQUSACE was Mr. David B. Mathis.

Dr. Robert M. Engler was Manager, EEDP, and Mr. Thomas R. Patin was the DOTS Program Coordinator. Dr. Raymond L. Montgomery was the Task Area Principal Investigator. The report was prepared by Dr. Marian E. Poindexter-Rollings under the general supervision of Dr. John J. Ingram, Chief, Water Resources Engineering Group (WREG); Dr. Montgomery, Chief, Environmental Engineering Division (EED); and Dr. John Harrison, Chief, EL.

This report constitutes a major portion of Dr. Poindexter's dissertation that was submitted to Texas A&M University in partial fulfillment of the requirements for the degree of Doctor of Philosophy. Acknowledgment is made to Mr. Mark A. Howard and Dr. Ronald E. Benson, WREG, for assistance in computer program modification. Laboratory testing assistance was received from the Soils Testing Facility, Soil Mechanics Division, Geotechnical Laboratory, WES, and from the Geotechnical Engineering Faculty of the US Military Academy, West Point, NY. Assistance in data presentation was provided by Mrs. Cheryl M. Lloyd, WREG. Technical reviewers for the report were Dr. Michael R. Palermo, Mr. Donald F. Hayes, and Dr. Benson of the EED. The report was edited by Mrs. Jessica S. Ruff of the WES, Information Technology Laboratory.

Commander and Director of the WES was COL Larry B. Fulton, EN. Technical Director was Dr. Robert W. Whalin.

This report should be cited as follows:

Poindexter-Rollings, Marian E. 1990. "Methodology for Analysis of Subaqueous Sediment Mounds," Technical Report D-90-2, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

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CONVERSION FACTORS, NON-SI TO SI (METRIC)
UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic metres
feet	0.3048	metres
inches	2.54	centimetres
miles (US nautical)	1.852	kilometres
miles (US statute)	1.609347	kilometres
pounds (force) per square foot	47.88026	pascals
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
square miles	2.589998	square kilometres

METHODOLOGY FOR ANALYSIS OF SUBAQUEOUS SFDIMENT MOUNDS

PART I: INTRODUCTION

Background

1. Each year, approximately 450 million cubic yards* of sediment is dredged from the rivers, harbors, and ship channels of the continental United States. After these sediments are dredged, they must be placed in an environmentally acceptable manner at designated disposal sites. Under present practice, approximately 40 percent of all dredged material is placed in upland (confined) disposal sites, while the remaining 60 percent is placed in subaqueous (open-water) disposal sites. With recent changes in environmental emphasis and developments in disposal technology, it is anticipated that pressure to increase subaqueous disposal will occur in the near future.

2. Placement of dredged material in an unconfined subaqueous disposal site typically results in formation of a mound of material on the floor of the water body. The sediments are often dredged by clamshell, placed in a bottom-dump barge, transported to the designated disposal site, and dumped through the water column; hopper dredges are also used to dredged and place material at subaqueous sites. The resulting mounds typically have side slopes of 0.5V:100H to 3.5V:100H.

3. As use of designated subaqueous disposal sites increases, the necessity of determining the holding capacity of these sites will increase. To obtain maximum use of the disposal sites, accurate account must be taken of the increase in storage capacity resulting from future decreases in the height of dredged material deposited. The height of the mound of material will decrease by one of two processes: consolidation and/or resuspension/erosion. The process of consolidation will be addressed herein, but the erosion process is not discussed in this report since it is being addressed in other work units being conducted by hydraulic modeling groups at the US Army Engineer Waterways Experiment Station (WES).

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 6.

4. The increases in the storage capacity of unconfined subaqueous disposal sites due to consolidation are especially important when these sites will be used to store large quantities of material from a number of dredging operations occurring over a period of years. Many soft fine-grained materials may undergo on the order of 50 percent vertical strain during the consolidation process. The objective is then to determine the amount of consolidation which the mound of dredged material will undergo as a result of self-weight consolidation.

5. Since the dredged material is very soft and can be expected to undergo significant amounts of consolidation, a finite strain theory of consolidation should be used to predict the consolidation behavior of the mounded material. Because existing subaqueous dredged material mounds have very flat slopes, a one-dimensional analysis of mound consolidation behavior will provide adequate results for either design or analysis of such mounds.

Purpose

6. The purpose of this report is to develop and document a procedure for predicting the consolidation behavior of subaqueous dredged material mounds. By establishing a methodology for analysis of subaqueous mounds, information can be obtained concerning the increases in site capacity which can be expected to occur over time. Incorporation of strength gain predictions will provide needed information for resuspension/erosion hydrodynamic models when fine-grained materials are involved.

PART II: LITERATURE REVIEW

7. Navigable waterways have performed, and will continue to perform, a significant role in the economic development and fiscal health of the United States. Responsibility for maintenance, improvement, and extension of these waterways was given to the US Department of the Army (and subsequently assigned to its Corps of Engineers) by the Rivers and Harbors Act of 1899. In fulfilling its mission, the Corps is responsible for the dredging and disposal of large volumes of sediment each year. Dredging is the process by which sediments are removed from the bottom of streams, rivers, lakes, and coastal waters; transported by one of several possible modes (ship, barge, or pipeline); and discharged onto land or into water. Approximately 450 million cu yd of dredged material is removed annually from the Nation's waterways. Of this quantity, about 355 million cu yd (or 79 percent) is removed through maintenance dredging operations, and about 95 million cu yd (or 21 percent) is new work dredging. Costs for annual dredging operations, at $1.46 per cubic yard in 1988 (Murden 1989), now exceed $657 million.

8. The portion of this dredging effort classified as maintenance dredging involves removal of recently deposited sediments from existing navigation channels; thus, the existing authorized channel depths are maintained in navigable waterways. The remainder of the dredging activity, classified as new work dredging, involves removal of sediments or residual bed materials from below the existing channel depths or from waterways that have not previously been dredged. With the anticipated passage of legislation authorizing port-deepening projects in the United States, the amount of new work dredging is expected to increase significantly in the near future. For instance, at Norfolk, VA, and Mobile, AL, planned channel-deepening projects are expected to generate approximately 46 and 184 million cu yd, respectively, of additional new work dredged material (Edgar and Engler 1984).

Disposal Alternatives

9. After sediments are dredged, they must be placed in an environmentally acceptable manner at designated disposal sites. Types of disposal sites that may be used include confined (intertidal, nearshore, and upland) disposal sites and open-water (aquatic) disposal sites. The various types of dredged

material disposal sites are illustrated in Figure 1. Typically the nearshore/intertidal/wetland disposal sites are initially constructed as confined disposal sites; the dikes are later breached or removed completely to allow tidal flushing of the area. These sites are therefore typically considered as confined disposal sites. At present, confined disposal sites receive about 193 million cu yd of dredged material, while open-water sites receive about 289 million cu yd annually.

10. Confined disposal sites (also termed confined disposal areas, containment areas, diked disposal areas, and confined disposal facilities) have been used extensively throughout the United States for many years. The majority of the maintenance dredged material along the Atlantic and Gulf coasts, including that from the major ports of Baltimore, Norfolk, Charleston, Savannah, Mobile, Galveston, and Houston, is placed in confined disposal sites. Confined disposal facilities are also used extensively for containment of the material dredged from numerous harbors along the Great Lakes. Confined disposal is sometimes required because contaminants in the maintenance sediments make them unsuitable for open-water disposal.

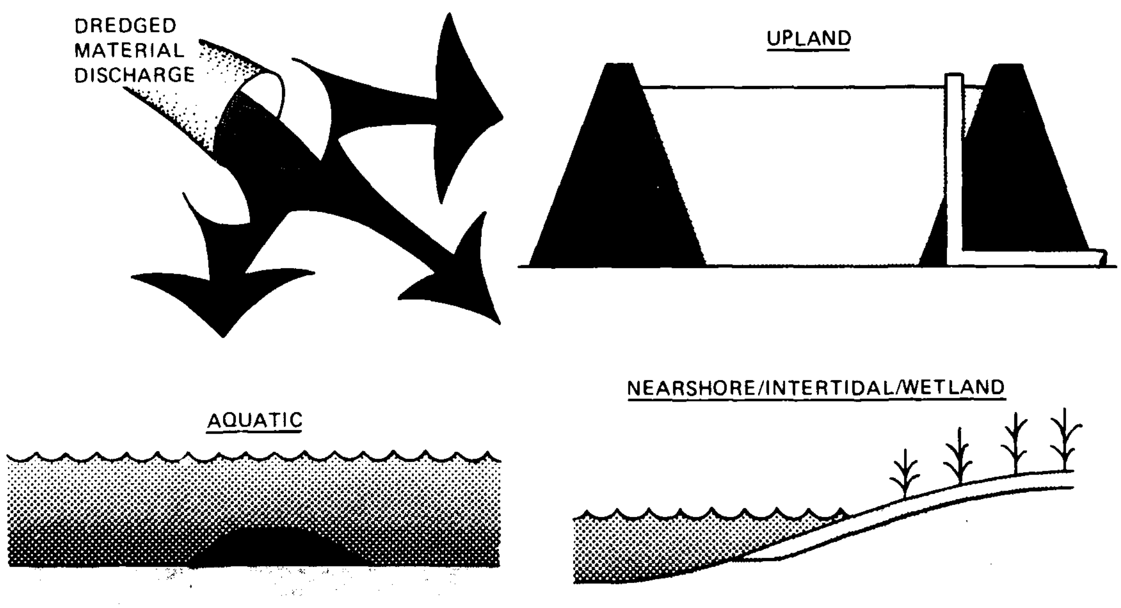

Figure 1. Types of dredged material disposal sites

11. The use of confined disposal sites increased significantly during the 1960s and 1970s as public concern heightened over open-water disposal of various waste materials, including dredged material (Francingues et al. 1985). Specifically of concern at this time were the environmental consequences of open-water disposal of contaminated dredged material. With this concern began a trend toward placement of the contaminated material in confined disposal sites.

12. Confined disposal sites were investigated extensively by the WES during the Dredged Material Research Program (DMRP) and the subsequent Dredging Operations Technical Support (DOTS) Program. Comprehensive engineering design and analysis procedures were developed during the DMRP and have been improved and verified by continuing field monitoring efforts under DOTS (Haliburton 1978; Montgomery et al. 1978; Palermo, Montgomery, and Poindexter 1978; Willoughby 1978; Hammer 1981; Palermo, Shields, and Hayes 1981; Cargill 1983, 1985; Headquarters, US Army Corps of Engineers 1987; Poindexter et al. 1988; Poindexter and Walker 1988).

13. Open-water disposal sites (also termed aquatic disposal sites, subaqueous disposal sites, and lake disposal sites (in the Great Lake)) have been used in some regions of the country for a number of years, although their use was severely curtailed for a period of 10 to 15 years because of environmental concerns and the resultant regulatory constraints. As more information is gained on the environmental effects of open-water disposal of dredged material and as new technology permits safer aquatic disposal of these materials, an increase in the use of this disposal option is expected. Not only is water transportation of dredged material relatively inexpensive, but the increasing scarcity of land available for development of upland, nearshore, and intertidal disposal sites often causes these disposal options to be prohibitively expensive.

14. While advances in both dredging and disposal technology are beginning to make the handling of contaminated sediments practical, it is the new disposal technology that is resulting in increased use of open-water sites for disposal of dredged material, including contaminated dredged material. (When contaminated dredged material is placed at an open-water disposal site, the site selected would obviously be a retention site, not an erosional site.) Two particular disposal methods can be attributed with the increased safety of open-water disposal: (a) level-bottom capping of contaminated sediment mounds

and (b) contained aquatic disposal (CAD). While both of these methods are sometimes loosely referred to as contained aquatic disposal, they will be referred to separately in this document. Although technology exists for design and construction of capped dredged material deposits, experience with capping is limited and the concept is still evolving. At present, capping projects have been successfully used in the United States, Japan, and Europe.

15. Capping of contaminated dredged material deposits involves controlled, accurate subaqueous placement of dredged material; isolation of the material from the receiving environment; and monitoring and maintenance of the site. Various combinations of materials, equipment, and techniques can be used to achieve the specific end products required at individual disposal sites. The level-bottom capping and CAD alternatives are illustrated in Figures 2 and 3, respectively.

16. Level-bottom capping involves placement of contaminated dredged material in a discrete mound on an existing level or gently sloping natural bottom. A confining cap of relatively uncontaminated (clean) material is then placed over the mounded material to prevent direct contact between the chemical contaminants and the overlying environment. The capping operation typically involves several discrete disposal sequences to provide adequate coverage of the contaminated material.

17. Contained aquatic disposal typically involves placement of contaminated dredged material in a bottom depression or confined subaqueous location. Types of locations that might be used include existing depressions, excavated depressions, former mining pits, or sites with constructed subaqueous confining dikes. The CAD alternative is typically used when the consistency of the material or the bottom topography of the disposal site requires positive lateral control, especially during the disposal operation. This disposal alternative may be used when heavily contaminated sediments must be dredged.

18. For either type of capped dredged material disposal, major considerations in the design process are the type and quantity of material needed to cover the contaminated material. These must be selected to ensure that the cap is capable of (a) controlling the movement of contaminants from the dredged material into the water column, (b) preventing direct contact between aquatic flora and fauna and the contaminated material, and (c) protecting the dredged material from erosion and transport away from the disposal site. Therefore, the capping material must act as a seal to isolate the contaminated

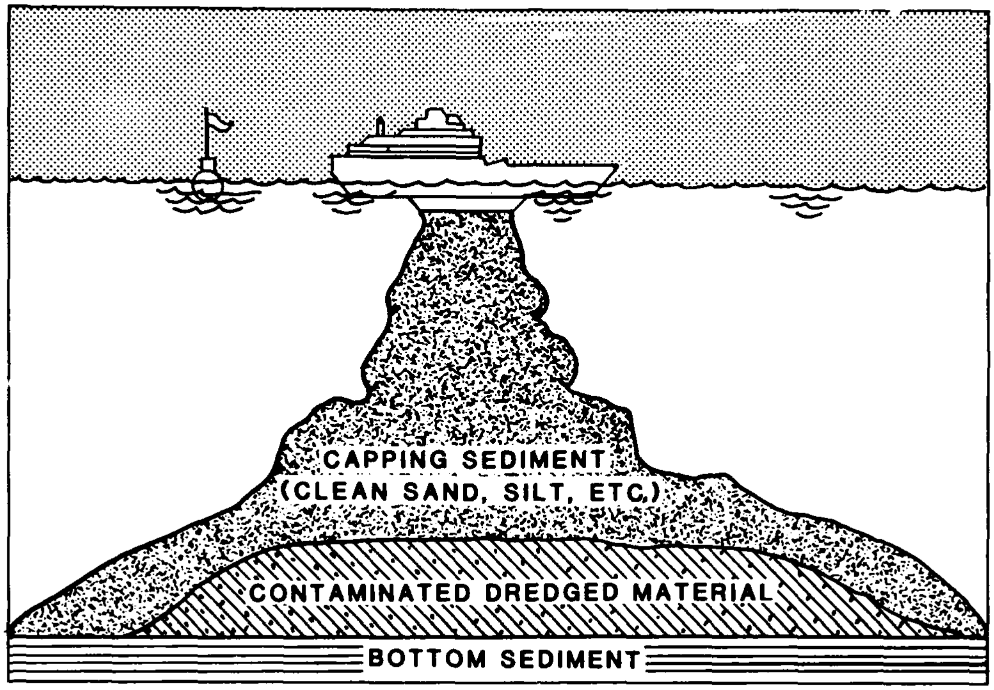

Figure 2. Schematic diagram illustrating the level-bottom capping concept

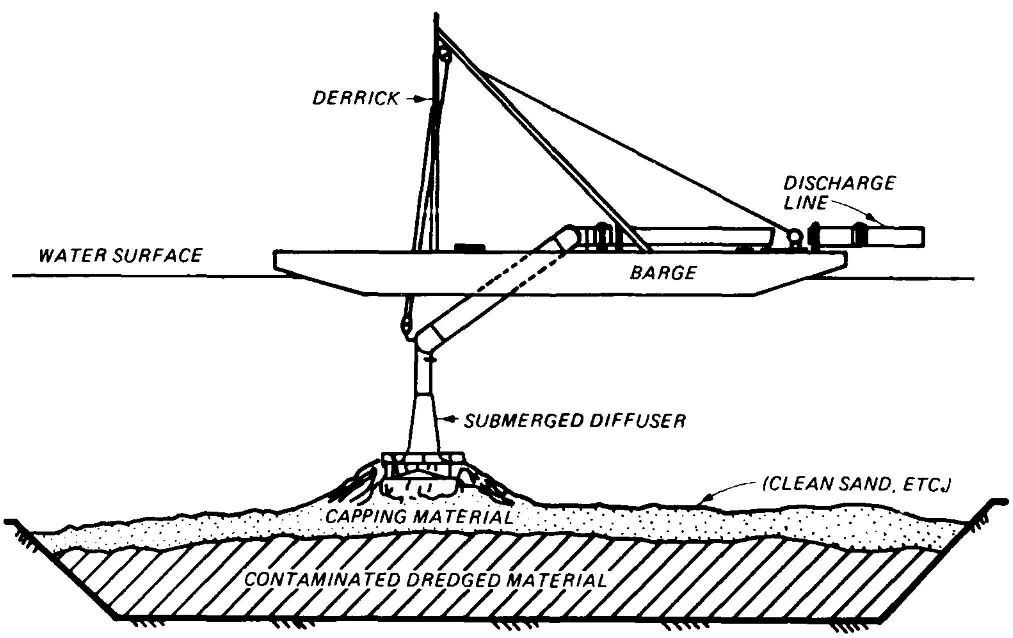

Figure 3. Schematic diagram illustrating the CAD concept and use of a submerged diffuser

material from the environment, and it must be resistant to resuspension and transport by the bottom shear stresses at the disposal site. Therefore, evaluation of consolidation is especially important for determining capping feasibility.

Designated Ocean Disposal Sites

19. While the major portion of the Ocean Dumping Act addresses the material being deposited at an ocean disposal site, Section 102 of MPRSA deals with the disposal site itself. This section requires that permanent disposal sites be designated to receive the various waste materials that will be deposited in the ocean. It also prohibits the approval of permanent ocean dumping sites without prior studies of the disposal site and without prior consideration of the Ocean Dumping Criteria published by USEPA (Federal Register, Vol 42, No. 7 (11 January 1977), pp 2476-2489).

20. Joint technical guidance has been developed by the USEPA and the Corps (USEPA/USACE 1984) to provide consistency in the evaluation and designation of environmentally acceptable and operationally efficient dredged material disposal sites. The site designation process was developed for the identification and designation of new ocean disposal sites, but it can also be applied to the permanent designation of historical disposal sites that have been designated on an interim basis. The designation process consists of three phases. Phase I involves the identification of the general area that is to be considered for disposal site designation and the collection of environmental data from the area. Phase II requires identification of specific candidate sites within the general area, and Phase III encompasses evaluation of the candidate sites and selection of one or more sites for permanent designation as open-ocean dredged material disposal sites.

21. Because sites could not immediately be designated as permanent disposal sites, Section 102 of MPRSA allowed the interim designation of historical disposal sites, and the USEPA and the Corps were allowed 3 years in which to complete the required site studies on the interim dredged material ocean disposal sites. Because the studies were not completed within 3 years, the USEPA extended the interim designation of 130 ocean disposal sites (Dortch et al., in preparation).

22. The interim dredged material disposal sites are generally located in proximity to the navigation channels requiring maintenance dredging and in areas that minimize their impact on navigation. A number of these disposal sites are located at historical disposal sites. The physical characteristics of all 130 interim disposal sites have been documented (Pequegnat et al. 1981).

Types of Dredging Equipment

23. The types of dredging equipment now used in the United States vary widely, but can generally be divided into two major categories: hydraulic and mechanical dredges. Hydraulic dredges include the hydraulic pipeline cutterhead dredge, the dustpan dredge, and the self-loading hopper dredge. Mechanical dredges include the dipper dredge, ladder dredge, and three types of bucket dredges: clamshell, orange-peel, and dragline dredges (Headquarters, US Army Corps of Engineers (HQUSACE) 1983; Hayes, Raymond, and McLellan 1984; Dortch et al., in preparation). Additionally, a number of special-purpose dredges have been developed in the United States and abroad to permit dredging of highly contaminated sediments at near in situ densities and/or to minimize the resuspension of sediments during dredging.

24. The type of dredging equipment used for a particular dredging project depends upon a combination of factors. Consideration must be given to the depth and location of sediments to be dredged, as well as to the level of contamination of the sediment. The type and quantity of material to be dredged are also major considerations in selection of dredging equipment. Additionally, the distance to the disposal site and the type of disposal must be considered. The types of dredges available and the required rate of production also influence dredge selection.

25. Other than the special-purpose dredges that are generally unavailable and are typically not compatible with conventional transport and disposal equipment, the various types of dredging equipment are discussed below. The expected behavior of the dredged material is also mentioned.

Hydraulic pipeline cutterhead dredge

26. The hydraulic pipeline cutterhead suction dredge is the most commonly used dredging vessel. It is capable of economically excavating large quantities of material and pumping it long distances through a pipeline to upland or

aquatic disposal sites. Use of the cutterhead in soft, fine-grained material generates a turbidity cloud that can result in adverse environmental impacts, especially when contaminants are present in the sediments. If the cutterhead is removed, the cutterhead dredge becomes essentially a plain suction dredge.

27. The dredged material discharged from the pipeline at an open-water disposal site typically descends rapidly to the bottom. Any coarse-grained material and clay balls will typically settle and accumulate directly beneath the discharge point. The remainder of the fine-grained slurry descends rapidly to the bottom as a well-defined jet of high-density fluid. This material will form a circular or elliptical mound of fluid mud with very flat slopes. Approximately 1 to 3 percent of the material discharged from a hydraulic pipeline dredging operation will remain suspended in the water column as a turbidity plume (Johanson, Bowen, and Henry 1976; Barnard 1978).

Dustpan dredge

28. The dustpan dredge was developed by the Corps of Engineers to maintain navigation channels in uncontrolled, sometimes very shallow, rivers. It was designed primarily to dredge sand and gravel, which compose the major portion of the bed load in these rivers. The dustpan dredge is somewhat similar to a cutterhead dredge in configuration. It is a hydraulic suction dredge that has a widely flared dredging head, instead of a cutterhead.

29. Because dustpan dredges were developed for use in coarse-grained material, they are not efficient in dredging fine-grained material and cause considerable resuspension of this material. They are therefore not recommended for use in contaminated sediments or in regions where turbidity must be closely controlled. Since these dredges are used to excavate coarse-grained material, the discharged material will tend to descend to the bottom immediately upon discharge from the pipeline and form a mound or ridge of material with relatively steep slopes.

Hopper dredge

30. The hopper dredge is a self-propelled seagoing ship that is capable of both dredging and transporting sediments. A unique feature of the hopper dredge is its ability to excavate a channel while under way, thus minimizing the interference caused to other water traffic both during dredging and transport.

31. When the dredged material is discharged from a hopper dredge at the disposal site, it falls through the water column as a well-defined jet of

high-density fluid; some water is entrained during the descent. Upon impact with the bottom, some of the material immediately comes to rest while the remainder moves laterally as a bottom surge until the turbulence of the surge is sufficiently reduced to permit deposition of the material. The shape of the resulting mound of material is dependent upon the relative quantity of material that immediately comes to rest and that in the surge.

Dipper dredge

32. The dipper dredge is basically a barge-mounted power shovel that is used to excavate hard materials that cannot be removed by other types of equipment. Although the dipper dredge can be used to excavate most bottom sediments, it is typically not used to dredge these materials because the violent action of the equipment during dredging causes resuspension of large quantities of soft sediment. A very small number of dipper dredges are in use today because of their limited applicability. In fact, the first dipper dredge to be built since 1926 was recently constructed in Louisiana.* When completed, this dredge will be the largest dipper dredge in existence with a shovel capacity of 26 cu yd.

33. Since the dipper dredge is typically used to excavate hard material, disposal of the dredged material should result in formation of a discrete mound of material immediately below the discharge point, with little or no lateral spreading. The mound would have the shape of a typical stockpile of rock or rubble. It is possible that the disposed material may penetrate and/or displace some of the bottom material at the disposal site because of the relative consistencies and densities of the materials involved.

Ladder dredge

34. The ladder dredge consists of a chain or conveyer belt that is mounted around the ladder frame and to which are attached buckets for excavating material. The entire apparatus is mounted on a barge-type floating vessel. The ladder dredge is normally used to excavate coarse-grained materials, including sand, gravel, cobbles, and blasted rock. This type dredge has been used extensively in California for placer mining of gold and in Malaysia for tin mining.

* Personal Communication, 1986, Claudia Seligman, Southern Shipbuilding Corporation, Slidell, LA.

35. Because of the method of dredging, the ladder dredge is appropriate for use with coarse-grained materials but is not suited for dredging fine-grained materials. Because of the relatively large exposed surface area per volume of material dredged and the turbulence caused in the water column by preceding buckets, many of the fines are lost from each bucket. This factor precludes use of the ladder dredge with contaminated sediments. Since the ladder dredge is used to excavate coarse material, any mound of material formed by this dredge would typically have a discrete shape with little or no lateral spreading.

Bucket dredge

36. The bucket dredge consists of a digging bucket that is operated by a crane; the entire apparatus is mounted on a barge-type floating vessel. Different types of buckets can be used, including the clamshell, orange-peel, and dragline. These buckets can be easily interchanged to accommodate the varying operational requirements. Sediments dredged with a bucket are typically placed in a bottom-dump scow or a split-hulled barge for transport to the disposal site.

37. The bucket-dredged material placed at the disposal site may have the consistency of a slurry such as that in a hopper dredge. Alternatively, material dredged by clamshell may have large lumps of more dense, intact sediment which may reach the bottom of the disposal site in this condition. In either case, the dredged material descends to the bottom rapidly with only a small percentage of the material remaining in the water column. The height and slope of the resulting mound are dependent upon material type and consistency.

Subaqueous Behavior

38. The behavior of dredged material discharged from hopper dredges or barges into open water can generally be divided into four phases (Johanson, Bowen, and Henry 1976): (a) convective descent and collapse, (b) dispersion, (c) bottom transport and resuspension, and (d) consolidation. These phases of behavior are illustrated in Figure 4. Convective descent and collapse occurs immediately after release of the dredged material. It is during this phase that the material falls under the influence of gravity until it either impacts and spreads along the bottom of the disposal site or arrives at a level of neutral buoyancy, at which point descent stops and horizontal spreading

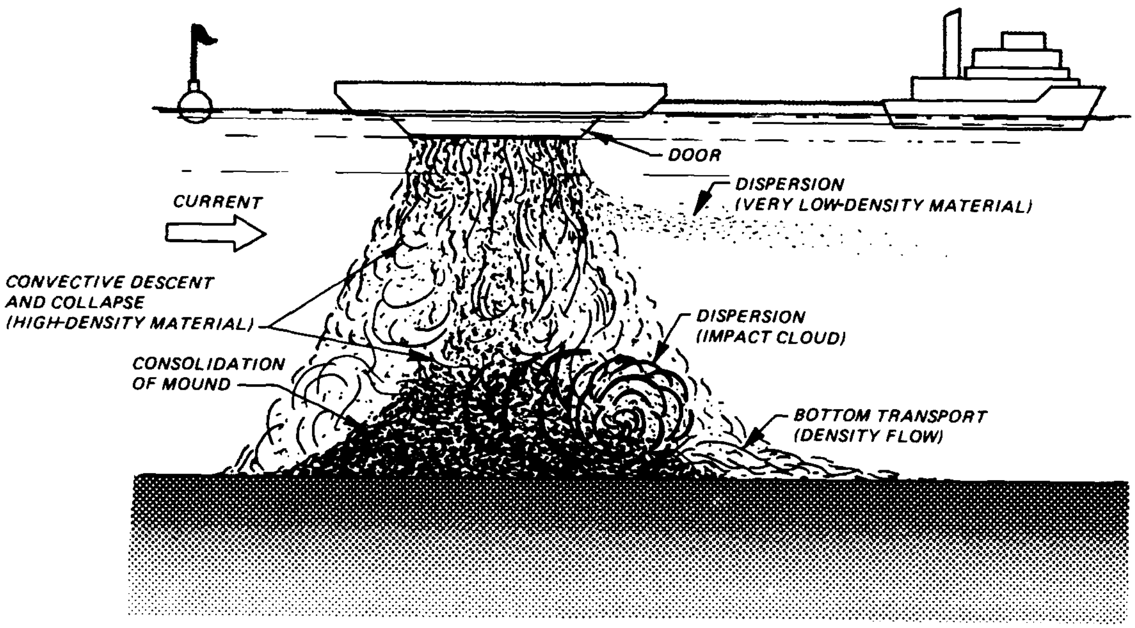

Figure 4. Four phases of dredged material behavior when the material is discharged subaqueously

begins. Dispersion is the process that consists of formation of a turbidity plume and movement of the suspended sediments remaining in the water column. Bottom transport and resuspension involves movement of the dredged material after it has been deposited on the bottom; the dredged material is then eroded by currents and may be swept outside the limits of the disposal site. The fourth phase, consolidation, occurs in both the mound of recently deposited dredged material and the underlying compressible bottom sediments.

39. Each of these phases of open-water dredged material disposal is affected by different processes. Among the factors influencing dredged material behavior at open-water disposal sites are the physical characteristics of the dredged material, such as its particle size distribution and mineralogical composition; the nature of the disposal operation, such as the type of discharge vessel, discharge rate, and solids concentration of the slurry; the physical environment in the vicinity of the disposal site, including currents, waves, tide, and storms; and bottom sediment characteristics and topography (Johanson, Bowen, and Henry 1976; Barnard 1978). The great variability of these factors from site to site, as well as potential seasonal fluctuations, increases the difficulty of predicting open-water dredged material behavior.

40. Reliable tools must be developed to allow evaluation of all potential physical impacts of open-water disposal, to guide field monitoring programs, to aid in disposal site selection, and to help resolve questions relating to dredged material disposal equipment and techniques. The first three phases of dredged material behavior are currently being investigated in joint efforts by the WES Hydraulics and Environmental Laboratories. Portions of the latter phase, consolidation, are the subject of this investigation, although the long-term behavior of dredged material mounds must consider both the consolidation and erosion of the mounded material.

Mound Characteristics

41. The major portion (80 to 95 percent) of dredged material dumped at open-water disposal sites reaches the disposal site bottom and forms a mound (Barnard 1978; Bokuniewicz et al. 1978; Nichols, Thompson, and Faas 1978; Tavolaro 1983; Truitt 1986; Dortch et al., in preparation). A typical mound configuration is shown in Figure 5. The configuration of a particular mound depends not only upon material type but also the method of dredging/disposal used. The dredge type affects the mound configuration by the amount of disturbance and especially the quantity of water introduced to the sediment during the dredging process. A hydraulic pipeline dredging operation completely remolds the sediment and introduces large volumes of water during the

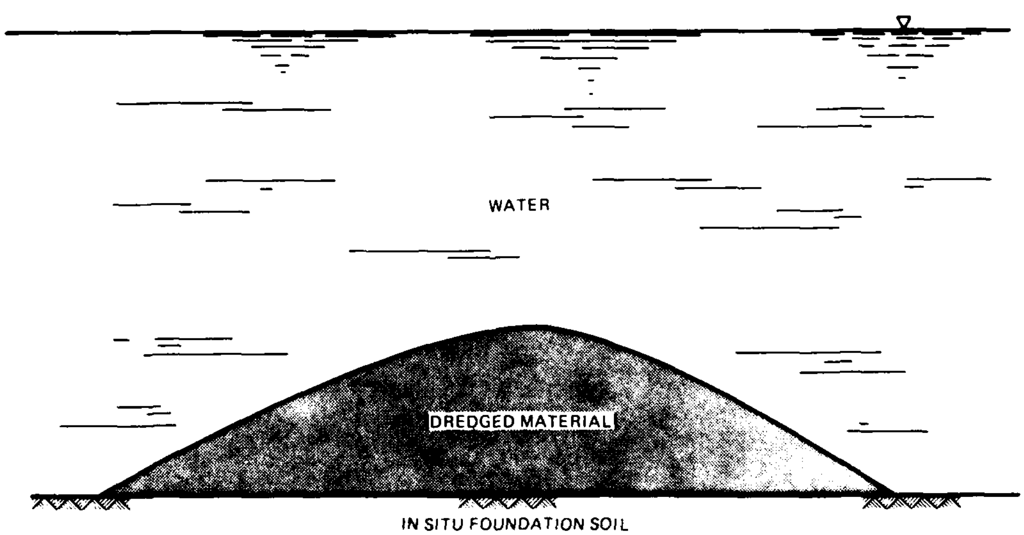

Figure 5. Typical mound configuration

dredging process. Typically the concentration of slurry discharged by a hydraulic dredging operation is approximately 145 g/ℓ (equivalent to a void ratio of approximately 17); this material generally has a maximum slope of 0.36V:100 H when placed in a confined disposal site, and this slope could be expected to be flatter if disposal occurred at an unconfined open-water disposal site. Clamshell dredging typically removes sediments at approximately their in situ water content; this material usually has void ratios in the vicinity of 5 to 7 (440 to 330 g/ℓ). Typical slopes of mounds formed by clamshell dredged material range from 1V:100H to 3.5V:100H.

42. The type of material dredged also affects the configuration of the mound (Bokuniewicz et al. 1978; Nichols, Thompson, and Faas 1978; Wright 1978; Demars et al. 1984; Dortch et al., in preparation). Hydraulically dredged fine-grained material will form very flat mounds with smooth topography. Fine-grained, cohesive sediments that have maintained some of their predredging structure (i.e., those sediments that have been mechanically dredged) will tend to form steeper mounds of smaller areal extent. These mounds will generally have very rough, uneven surfaces. Conversely, coarse-grained, noncohesive materials tend to form somewhat flatter mounds that have smoother surfaces, regardless of the dredging method used. This occurs as a result of the effective "raining down" of the noncohesive material through the water column. Thus, the combined influence of dredging method and material type on mound configuration is more pronounced with fine-grained material (see Table 1).

Site Capacity

43. With the approval of the London Dumping Convention and the concurrence of the US Congress that ocean disposal of dredged material could be conducted in a safe and environmentally acceptable manner, it is anticipated that a significant increase in subaqueous disposal of dredged material will occur in future years. Although considerable research was conducted during the DMRP concerning open-water disposal of dredged material, many aspects of this multifaceted technique require further, in-depth evaluation.

44. One area of particular concern is that of subaqueous disposal site capacity. Disposal site capacity can roughly be defined as that quantity of material that can be placed within the legally designated disposal site

Table 1

Initial Conditions Expected in Mounds

Type of Dredge	Sediment Dredged	Range of Void Ratio of Dredged Material	Site Slopes of Mounds
Hydraulic pipeline cutterhead	Clay Silt Sand Gravel Soft rock	17-18	0.36V:100H(max.) for fine-grained soil; otherwise, angle of repose
Dustpan	Sand Gravel	Same as in situ or slightly higher	Discrete mound; minimal lateral spreading
Hopper	Clay Silt Sand	15-18	0.5V:100H (max.) for fine-grained soil; otherwise, angle of repose
Dipper	Hard material (stiff clay, rock, shell, coral reef)	Same as in situ	Discrete mound; little or no lateral spreading
Ladder	Coarse-grained material	Same as in situ	Discrete mound
Bucket			
clamshell	Clay Silt Sand	Same as in situ	1.0V:100H to 3.5V:100H
Orange-peel	Rock	Same as in situ	Discrete mound; little or no lateral spreading

without extending beyond the site boundaries, interfering with navigation, or extending to heights that would subject it to unacceptable levels of sediment resuspension from wave action. In addition to physical aspects, both biological and chemical effects of disposal site capacity must be considered. Generally, the physical deposition and movement of dredged material will determine the biological and the chemical effects of dredged material disposal because of the physical location of the dredged material and the association of chemical contaminants with the dredged material particles. If the biological and chemical effects reach unacceptable levels, the amount of dredged material to be placed at a designated disposal site might have to be reduced. Thus, the biological and chemical aspects of site capacity may be thought of as factors that impact but do not generally control ultimate disposal site capacity.

45. To determine the capacity of subaqueous dredged material disposal sites, two aspects of dredged material placement and behavior must be investigated. The shapes and side slopes of dredged material deposits resulting from subaqueous disposal of dredged material must be better defined. This will require field monitoring of a number of constructed subaqueous deposits to better define the effect of the method of dredging, type of dredged material, depth of water through which the dredged material descended, and type of disposal site used on the resulting deposit. After the dredged material has been placed in a subaqueous disposal site, the behavior of the mounded material must be investigated. The behavior of interest includes both consolidation and erosion of the deposited dredged material. Work conducted as a part of this investigation is intended to address the consolidation behavior and the resultant increase in shear strength of the material. Erosion will not be addressed directly, although the shear strength predictions will form the basis for future investigations into erosion potential; additionally, the shear strength data can be used in some of the existing hydraulic models that attempt to predict the erosion potential of subaqueous dredged material disposal mounds (Williams 1983; Dortch et al., in preparation).

PART III: METHODOLOGY OF ANALYSIS

46. A systematic procedure to analyze the behavior of subaqueous dredged material mounds was developed as a part of this investigation. For this methodology of analysis to be complete, it must include not only the physical/engineering considerations, but also the chemical and biological aspects associated with dredged material disposal. By including all aspects, this procedure provides the necessary framework for successfully analyzing the effect of dredged material disposal on subaqueous disposal site capacity. A flowchart depicting this procedure is shown as Figure 6.

47. In the following paragraphs, the general methodology for analysis is discussed. Subjects such as characterization of the dredged material, selection of a disposal site, selection of operational methods, and investigation of the engineering behavior of the mound are included as initial steps in the methodology. Using the information gained from these investigations, mound behavior can be predicted and compared with project requirements to determine the acceptability of the proposed disposal mound. If the mound characteristics are acceptable, then disposal operations can proceed. After disposal has been completed, the mound should be monitored to verify that the predicted behavior accurately models the actual field performance. Necessary factors for consideration are included in all topical areas. The following sections provide more detailed information on the various engineering aspects of the methodology which are of interest in this investigation. Although this methodology is intended to be used for subaqueous disposal of dredged material, many portions of the procedure can easily be applied to other types of disposal operations.

Characterization of Sediment

48. Dredging projects are undertaken when a need arises to remove accumulated sediments from a water body. Therefore, the first item to be addressed is location of the sediment to be dredged. The quantity of material must be determined; this is usually done by use of hydrographic surveys. Representative samples of the sediment must then be collected and tested. The necessary tests include a suite of both chemical analyses and geotechnical characterization tests. The chemical, and possible subsequent biological testing, that

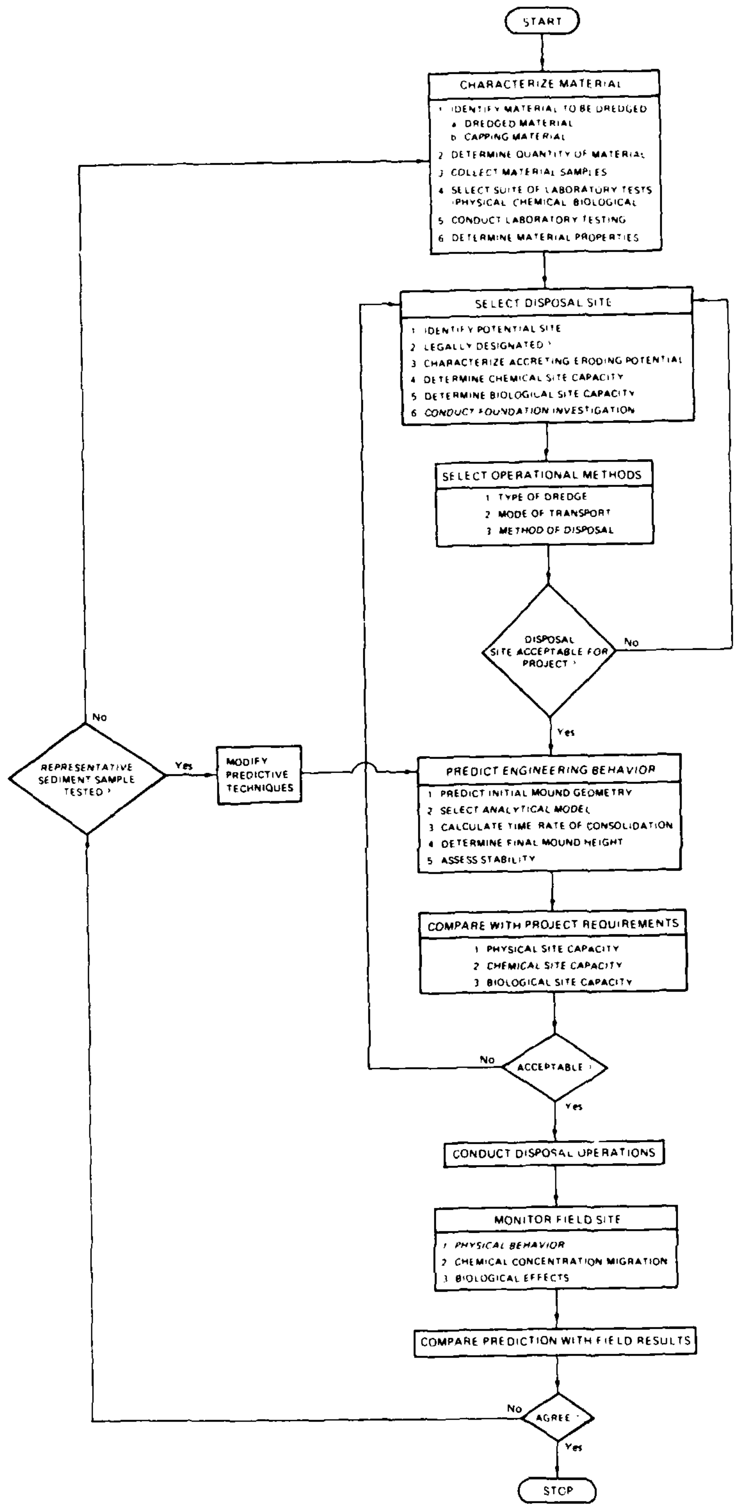

Figure 6. Flowchart depicting methodology of analysis

will be necessary will depend upon the type and amount of contaminants present in the sediment. Procedures for identifying the needed chemical and biological analyses are presented by Francingues et al. (1985). The geotechnical tests required include engineering classification tests, consolidation tests, and permeability determination; the rationale for selecting these tests is discussed by Poindexter (1987, 1988). Analysis of test results will provide information that can be used to assist in disposal site selection as well as any further evaluations.

Disposal Site Selection

49. An acceptable disposal site must be selected for every dredging operation. The disposal site must meet the established criteria for the specific region and project of interest. Initially, the site must be within reasonable proximity to the dredging site; the economics of transporting the dredged material by appropriate means must be evaluated. The proposed disposal site must also be a legally designated disposal site that has been approved by the USEPA.

50. The characteristics of the subaqueous disposal site must be compatible with the dredging project and dredging/disposal method to be used. The initial consideration is the erosive nature of the site; both dispersive and accreting sites are included in currently designated disposal sites. For disposal operations involving contaminated dredged material, the disposal site should be one with very low or nonexistent erosive forces. In certain instances, such as disposal of uncontaminated sediments from the San Francisco Bay area, the dredged material may purposefully be placed in a dispersive site so that the material will be swept to sea, thus maintaining the capacity of the disposal site for future use.

51. When an accreting site is to be used and the dredged material will remain onsite, the topography of the disposal site (surface slope and/or size and shape of depressions) must be considered with regard to the level of confinement of dredged material which will be required. The necessary level of confinement is dependent upon the type of material, its level of contamination, and the dredging method used. After the initial considerations have been satisfied for a particular disposal site, a foundation investigation should be conducted to determine the geotechnical properties of the foundation

soils. This information will provide the necessary compressibility and permeability to determine the interactive effects, such as drainage and settlement, of the foundation soil and the dredged material. It should be noted that a foundation investigation is not normally required for erosive disposal sites.

Operational Methods Selection

52. Operational methods selected for dredging and disposing of the material must be consistent with project requirements. The type of dredge, mode of transport, and method of disposal must all be selected. The type of dredge chosen will depend first upon the type of material to be dredged and then upon the amount of contamination involved and the in situ location of the sediments. The mode of dredged material transport to the disposal site will be determined by a combination of the following factors: the dredging method used, relative locations of the dredging and disposal sites, amount of contamination in the sediments, and the proximity of navigation channels to the operations. The disposal method chosen will depend on the mode of transport, conditions at the disposal site, and relative contaminant concentration in the dredged material and the water column at the site. Additional information concerning dredging, transport, and disposal operations was presented in Part II.

Engineering Behavior Prediction

Initial mound geometry

53. The expected geometry of the mound upon completion of the disposal operation must be obtained. The geometry may be estimated by using one of a number of hydraulic models that have been developed to predict the descent and spread of dredged material after release at an open-water disposal site. The refinement of the models and, conversely, the simplicity of use vary tremendously between models. The model most often used within the Corps of Engineers is the Johnson model (Brandsma and Divoky 1976; Johnson, in preparation). It is capable of simulating continuous, instantaneous, and sequenced disposal operations. The model requires a number of empirical coefficients for calibration; very little data exist for these coefficients, and validation

studies are still ongoing. The model is currently being modified for use on a microcomputer as a part of the Automated Dredging and Disposal Activities Management System (ADDAMS) (Walski et al. 1984). If a hydraulic model is not used, the mound geometry can be approximated empirically by using data from existing mounds and considering the method of dredging and the material type involved.

Mound consolidation

54. To use the engineering properties determined during the laboratory testing to analyze the potential disposal mound, an appropriate analytical model must be selected; this involves selection of a consolidation theory and a corresponding numerical computer model. The very soft nature of most dredged materials results in large strains; consequently, the conventional Terzaghi consolidation theory is inappropriate for use with these materials. The finite strain theory of consolidation is more appropriate and should be used since it can account for the large strains and nonlinear material properties of soft soils (Cargill 1982, 1985). A detailed discussion of these consolidation theories and selected numerical modeling is presented in Part IV. After the theory and model are selected, calculations should be made to determine the amount and rate of consolidation that will occur. Some determination or estimation of the shear strength of the deposit should also be made. This information can be used to evaluate the stability of the mound and to provide an indication of the mound's resistance to erosive forces at the disposal site.

Project Requirements Comparison

55. Results from the engineering behavior prediction should be used to evaluate the acceptability of the proposed mound with regard to project requirements. These requirements may vary widely depending upon the geographic region in which the disposal site is located, the circulation patterns near the site, size of the disposal site, and specific factors of the dredging and disposal operations. In all cases, the projected size of the disposal mound must be considered. The mound should be small enough with respect to the surrounding water body to avoid having any significant effects on the surrounding environment. The mound must not extend beyond the horizontal site boundaries. Both the initial and final height of the mound must be

considered with regard to the required depth for navigation and the depth of influence of both normal wave action and expected storms. The former consideration is necessary to prevent interference with navigation, while the latter is required to prevent resuspension and erosion of surficial material. Both the biological and chemical site capacity must be evaluated to ensure that the proposed mound will not exceed either of these criteria. This requires that the effect on surrounding water quality as well as the effect on biological resources must be within the prevailing legal standards. If the proposed mound will meet all regulatory criteria, then planning can proceed toward creation of the mound. If the criteria are not met, another site must be selected and the evaluation procedure repeated.

Dredging and Disposal Operations

56. Actual dredging/disposal operations must be tailored to the particular dredged material and disposal site. In many locations, these operations are limited to certain portions of the year to minimize their impact on the biological species in the area; such restrictions on the allowable "dredging/disposal window" prevent, for example, disposal during periods of upstream salmon migration and during movement of oyster larvae. Disposal operations are also significantly affected by the level and type of contaminants present in the sediment. Care must be taken when dealing with contaminated sediments to minimize suspension of particles in the water column during both dredging and disposal. In cases where the sediments are highly contaminated, such as at USEPA Superfund dredging sites, especially designed and fabricated equipment may be required to eliminate the unnecessary resuspension of contaminated particles during both dredging and disposal operations.

57. When dredging and disposal operations involving contaminated sediments occur, monitoring is typically conducted before and after dredging to ensure that all of the contaminated material that must be dredged is removed during this one operation. During disposal operations, monitoring is conducted periodically at the disposal site to verify the location of various discharges of dredged material. This typically initiates a long-term monitoring effort at these sites.

Field Site Monitoring

58. To determine the long-term fate of subaqueous mounds, field site monitoring should be conducted to monitor the postdisposal behavior of dredged material mounds. By observing the mounds over a period of time, the adequacy of the assumptions made and the mound design can be verified. If problems are observed at a particular mound of contaminated material, remedial measures can be undertaken at that mound to protect the environment from exposure to contaminants; the assumptions, procedures, and methods used in mound construction can also be refined and modified to provide more conservative designs in the future.

59. Hydrographic surveys are the most commonly used method of monitoring subaqueous dredging/disposal sites. These surveys provide coverage over a large area with minimal expense and time involved. Also, the equipment needed to conduct hydrographic surveys is usually available to Corps offices, the entities responsible for dredged material disposal, navigation, and subaqueous monitoring. Additional methods that may be used for monitoring include settlement plates to measure material settlement and core borings to provide profiles of engineering properties. Other remote methods that may be used include side-scan sonar, subbottom profiling, and various other settlement/pore pressure monitoring techniques. Any of these methods may be supplemented by diver observations. Several of these techniques were used at various disposal sites and are described in Part V of this report.

Prediction Versus Observation

60. As field data are collected over time, this information should be compared with the predicted mound behavior to verify the predictive procedure. If good agreement is obtained, it can be concluded that the predictive techniques utilized are appropriate for use at similar subaqueous disposal sites. Alternatively, if the predictions do not accurately model the observed field behavior, the data obtained from the field sites can be used to modify and refine the predictive techniques.

PART IV: DEVELOPMENT OF CONSTITUTIVE RELATIONSHIPS

Background

61. The theory of one-dimensional consolidation of saturated clays was first formulated by Karl Terzaghi (1921, 1923a, 1923b, 1924). Since initial formulation, this soil mechanics relationship has been used throughout the geotechnical community, although it consistently results in discrepancies, sometimes large, between predictions and field behavior. It is recognized that these discrepancies result from the use of simplifying assumptions in the theoretical formulation since the assumptions are only approximately satisfied in nature. Because of the lack of accuracy resulting from use of the Terzaghi consolidation theory, considerable effort has been expended on modification and improvement of it.

62. The governing equation for the Terzaghi consolidation theory is typically written as (Terzaghi 1942)

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial \bar{z}^2} \qquad (1)$$

where

u = excess pore water pressure

t = time

c_v = coefficient of consolidation

$\bar{z}$ = vertical space coordinate

The physical assumptions involved in the theory are as follows: (a) the soil is completely saturated, (b) the soil particles and the pore water are incompressible, (c) Darcy's law is valid, (d) the coefficient of permeability k of the soil mass is constant, and (e) the rate of consolidation is due entirely to the low permeability of the soil, i.e., the process is one of primary consolidation. Additionally, the coefficient of compressibility, a_v, which is assumed to be constant, is defined as

$$a_v = -\frac{\Delta e}{\Delta \sigma'} \qquad (2)$$

where

Δe = change in void ratio during consolidation

$\Delta \sigma'$ = change in vertical effective stress during consolidation

63. Implicit in the above assumptions are several additional assumptions. Since it is stated that the fluid flow is governed by Darcy's law and this law does not consider the relative velocity of the fluid and solid phases, it is generally assumed that the velocity of solids is zero. It is also assumed that k and a_v [and thus m_v, since $m_v = a_v/(1 + e_o)$] are constants. Since m_v is based upon the initial void ratio e_o, the assumption is implicitly made that the consolidation theory is based upon the concept of engineering strain; this implies that the strain in the soil mass will be small, i.e., infinitesimal strain will occur.

64. Numerous modifications have been made to the original one-dimensional consolidation theory in order to more closely describe field behavior. These changes are typically intended to address one of three general areas of concern: multidimensional geometry, nonlinear material properties, and magnitude of strain. The first concern addressed is that the theory is one-dimensional while the phenomenon occurring in the field is three-dimensional. The extension to a multidimensional geometry was accomplished by Rendulic's (1936) theory in which the magnitude and progress (rate) of settlement are not coupled and by Biot's (1935, 1941, 1955, 1956) coupled theory. It is generally realized that a coupled multidimensional consolidation theory should be used instead of either the one-dimensional or the uncoupled three-dimensional theory (Schiffman, Chen, and Jordan 1969).

65. The second improvement in the theory resulted from the realization that linear constitutive relationships do not adequately model field behavior and may result in large errors, especially if large changes in void ratio occur during consolidation. In the Terzaghi and Biot theories, both the permeability and compressibility of the soil are assumed to remain constant during consolidation under a particular load increment. The classical Terzaghi theory has been extended (Richart 1957, Lo 1960, Schiffman and Gibson 1964, Barden and Berry 1965, Davis and Raymond 1965, Janbu 1965) to consider the variation of permeability and compressibility during consolidation. It should be noted that while variations in these parameters will be most significant when strains are large, the modified theories are invariably based upon infinitesimal strain theory.

66. The third improvement involves utilizing finite strain consolidation theory instead of the typically used infinitesimal strain theory. The general framework for the extension of the theory was developed by Ortenblad (1930) and McNabb (1960). Numerous publications present the various finite strain formulations (Mikasa 1963, 1965; Davis and Raymond 1965; Gibson, England, and Hussey 1967; Simons and Beng 1969; Berry and Poskitt 1972; Mesri and Rokhsar 1974; Monte and Krizek 1976; Lee 1979; Lee and Sills 1979, 1981; Gibson, Schiffman, and Cargill 1981). Analysis of these theories has shown that the most general formulation of the finite strain consolidation theory is that developed by Gibson, England, and Hussey (1967); all other formulations can be considered to be special (restrictive) cases of this one. The Mikasa theory differs from the Gibson, England, and Hussey (1967) theory only in the initial condition (Pane 1981, Pane and Schiffman 1981). The relationship between the various finite and infinitesimal strain theories has been discussed by Schiffman (1980).

67. It is interesting to note that the original Terzaghi formulation has been shown to be a finite strain consolidation theory (Znidarcic 1982) that was reduced to the classical infinitesimal theory when the governing equation was derived in Lagrangian coordinates (Terzaghi and Frohlich 1936). Although no reason was given by Terzaghi or Frohlich for the change in coordinates, Ralph B. Peck (in a 1981 letter to Robert L. Schiffman) proposed a possible explanation:

> My hunch, and it is little more than that, is that Terzaghi lost interest in the theory of consolidation as a theory once he had originally developed it far enough to see its significance, and had judged its applicability and shortcomings on the basis of laboratory tests. He was not, as he was the first to admit, a theorist beyond the extent that seemed to be necessary to understand the behavior of earth material. Frolich, on the other hand, was much more theoretically minded. It might well be that, in Vienna, when Frolich and Terzaghi were cooperating on their book, Terzaghi was more than happy to let Frolich elaborate on and manipulate the theory as much as he desired. Possibly the change in coordinates came through Frolich.

68. An additional factor not considered in the classical Terzaghi theory but which should be included in the consolidation theory is the self-weight of the consolidating layer. When the self-weight of the material is the only force causing consolidation or when the self-weight stresses are comparable (equal in magnitude) to the externally applied stresses, the weight of the deposit should definitely be considered in any analysis. The Gibson, England,

and Hussey (1967) theory accounts for the material's self-weight and can be used to analyze cases of rapid sedimentation (Pane and Schiffman 1981), slow sedimentation (Schiffman and Cargill 1981), and loaded clay layers (Gibson, Schiffman, and Cargill 1981).

Finite Strain Consolidation

69. The Gibson, England, and Hussey (1967) theory of finite strain consolidation can accommodate most of the concerns discussed previously. This theory places no restriction on the magnitude of strain. It also makes no assumption concerning the material's constitutive relationships; point data are used to define the laboratory-determined relationships. The self-weight of the material is included in this theory. Only one area of concern, that of the dimensionality of the problem, is not presently addressed by the Gibson, England, and Hussey one-dimensional theory.

70. The physical assumptions required for development of the finite strain consolidation theory are similar to those of the Terzaghi infinitesimal strain theory. The assumptions are as follows:

- a. The soil system is saturated and consists of soil particles and pore fluid; the soil particles form a compressible soil matrix.
- b. The soil particles and the pore fluid are incompressible.
- c. The soil skeleton deforms in either a linear or nonlinear manner with no restriction on the magnitude of strain.
- d. The fluid is Newtonian, and its flow through the porous skeleton is governed by Darcy's law.
- e. The fluid flow velocities are small.

71. Because the magnitude of strain is not restricted, the thickness of the consolidating layer changes with time and becomes a variable of the problem. Thus, the Lagrangian coordinate system, typically used in geotechnical engineering, which is fixed in time and space, cannot be used. Instead an Eulerian system is used in development of the governing equation since the convective coordinates are functions of time. To facilitate the mathematical analysis, the material (reduced) coordinate system is used for the analysis. This coordinate system is based upon the volume of soil particles contained between a datum plane and the point being analyzed (Ortenblad 1930, McNabb 1960). Use of the various coordinate systems is discussed in detail by Cargill (1982).

Governing Equation

72. Derivation of the governing equation for one-dimensional finite strain consolidation requires use of the balance laws as well as the constitutive relationships for the soil system. The balance laws required are the conservation of momentum (equilibrium) and the conservation of mass (continuity). Using these balance laws, equilibrium relationships can be established for the bulk mixture and the fluid phase, and continuity relationships can be developed for the fluid phase and for the solid phase. Constitutive relationships utilized include the effective stress principle, the flow relationship, and the material functions (both the void ratio-effective stress and the void ratio-permeability relationships). The governing equation will be derived in Eulerian coordinates.

73. The governing equation for one-dimensional finite strain consolidation can be developed in either Eulerian ξ, Lagrangian $\underline{a}$, or material z coordinates. Furthermore, it may be expressed in terms of either dependent variable: excess pore water pressure u or void ratio e. For this report, the Eulerian ξ coordinate system is used. and the equation is written in terms of void ratio. The governing equation for finite strain consolidation in Eulerian ξ coordinates with the void ratio as the dependent variable is

$$-\left(\frac{\gamma_s}{\gamma_w} - 1\right) \frac{d}{de}\left(\frac{k_\xi}{1 + e}\right)\frac{\partial e}{\partial \xi} - \frac{v_s}{1 + e} \frac{\partial e}{\partial \xi} + \frac{\partial}{\partial \xi}\left[\frac{k_\xi}{\gamma_w a_v}\right]\frac{\partial e}{\partial \xi} = \frac{1}{1 + e}\frac{\partial e}{\partial t} \qquad (3)$$

where

γ_s = unit weight of solids

γ_w = unit weight of fluid

k_ξ = coefficient of permeability in convective (ξ) coordinates

v_s = velocity of solids

t = time

It should be noted that the terms k_ξ and a_v are both dependent upon the void ratio, i.e., $k_\xi(e)$ and $a_v(e)$. The first term in Equation 3 accounts for the self-weight of the consolidating layer.

74. Solution of this equation requires a numerical procedure since the presence of nonlinear coefficients precludes analytical solution. By using a

computer, numerical solution of the governing differential equation is feasible. Material properties required include the specific gravity and the relationships between (a) void ratio and effective stress and (b) void ratio and permeability. These values should be determined from a laboratory testing program.

Initial Conditions

75. The initial void ratio distribution in a dredged material mound will depend upon the interaction of a number of factors, including the type of material (i.e., the unit weights of solids and fluids in the mound), condition before dredging, dredging method, the effective weight of any existing surcharge, and the relationship between void ratio and effective stress for the dredged material composing the mound.

76. The consistency or degree of consolidation of the material before dredging will depend upon the sediment type and the length of time that the sediment has been in this location, i.e., how much consolidation has occurred. When sediment is dredged, this consistency will be disturbed/disrupted; some amount of disturbance will result no matter what dredging procedure is used. Hydraulic dredging will completely remold the material. Mechanical dredging will cause some remolding; a portion of the material may initially remain at the same void ratio, although this may change as the stress conditions change.

77. Because of the disturbance of the sediments during dredging, it would be inappropriate to assume that immediately after disposal the material had a void ratio distribution resulting from the progression of self-weight consolidation. Therefore, it will be assumed that the dredged material mound is deposited instantaneously at a uniform consistency, and immediately after disposal, the material has a uniform void ratio throughout its depth. Thus, the initial void ratio, e_o of an instantaneously deposited mound may be expressed as

$$e_o = \text{constant with depth} \tag{4}$$

The numerical value of the initial void ratio existing throughout a dredged material mound will depend upon the particular sediment involved and the type of dredging equipment used.

78. In an existing mound, the dredged material will most likely have undergone self-weight consolidation. In this case, the initial conditions would be expressed as

$$e = f(\sigma') \tag{5}$$

and the void ratio at any depth is found from the $e - \sigma'$ relationship for that material. If a surcharge load exists, the effective stress used to determine the void ratio must include both the weight of any overlying dredged material and the surcharge load.

Boundary Conditions

79. Three boundary conditions are possible for a compressible dredged material mound: (a) a free-draining boundary, (b) an impermeable boundary, or (c) a semipermeable boundary (Cargill 1982). These boundary conditions may apply at either the top surface of the mound or the bottom of the dredged material deposit; different boundary conditions may apply to the top and bottom of the mound.

80. When a free-draining boundary exists, no excess pore water pressure exists at the boundary. Thus,

$$u = 0 \tag{6}$$

The total pore pressure, u_w, will then be equal to the static pore pressure, u_o,

$$u_w = u_o = h_w \gamma_w \tag{7}$$

where h_w is the height of the water table above the boundary. The effective stress, σ', at the boundary may be obtained by calculating the total stress at that point and applying the effective stress principle, such that

$$\sigma' = \sigma - u_w = \sigma - h_w \gamma_w \tag{8}$$

where σ is the total stress and is equal to any applied load plus the self-weight of the material. The void ratio at the boundary is then obtained by using the known $e - \sigma'$ relationship for the dredged material,

$$e = f(\sigma') \tag{5,bis}$$

81. For the case of an impermeable boundary, there is no fluid flow across the boundary; therefore, the velocity of fluid is equal to the velocity of solids,

$$v_w = v_s \tag{9}$$

and

$$\frac{\partial u}{\partial \xi} = 0 \tag{10}$$

After appropriate mathematical development (Cargill 1982, Poindexter 1988), the following equation is obtained

$$\frac{\partial e}{\partial \xi} + \frac{\gamma_s - \gamma_w}{(1 + e)\,\frac{d\sigma'}{de}} = 0 \tag{11}$$

This is the boundary condition, expressed in Eulerian ξ coordinates, in the compressible material at an impermeable boundary.

82. In some cases, a boundary may be encountered which is semipermeable. This boundary condition is based upon the assumption that the quantity of fluid flowing out of one layer must equal the quantity of fluid flowing into the adjacent layer across the semipermeable boundary. Therefore, the following relationship can be developed in material, z, coordinates:

$$\left(\frac{k}{1 + e}\,\frac{\partial u}{\partial z}\right)_T = \left(\frac{k}{1 + e}\,\frac{\partial u}{\partial z}\right)_B \tag{12}$$

where the subscripts indicate the upper (T) and lower (B) layers. Since both the total and static pore water pressures must be equal in the two layers at the common boundary, then

$$(u)_T = (u)_B \tag{13}$$

83. To obtain an expression for the void ratio at a semipermeable boundary, the effective stress principle, equilibrium of the mixture, and equilibrium of the fluid are used to obtain the expression

$$\frac{\partial e}{\partial z} = \left(\gamma_w - \gamma_s - \frac{\partial u}{\partial z}\right) \frac{de}{d\sigma'} \tag{14}$$

By using Equations 12-14, the numerical problem of semipermeable boundaries can be solved.

Numerical Solution of Equation

Explicit finite difference procedure

84. Solution of the governing equation may be achieved by replacing the continuous derivatives contained in the equation by the ratio of the changes in the variable over a small (finite) increment. Thus, the differential equation can be changed into a difference equation. Difference approximations must be written for both the first and second derivatives which are contained in the governing equation. For ease of computation, the governing equation will be solved in material, z, coordinates. The transformation between convective ξ and material coordinates is

$$\frac{\partial \xi}{\partial z} = 1 + e(z,t) \tag{15}$$

85. To develop the appropriate difference approximations, both time and space are divided into discrete increments of τ and δ, respectively, as shown in Figure 7. By expanding discrete points into a Taylor series, the expressions for the necessary derivatives may be obtained. The time derivative of the void ratio is expressed as

$$\frac{\partial e}{\partial t}(z_i, t_j) \simeq \frac{1}{\tau}(e_{i,j+1} - e_{i,j}) \tag{16}$$

where the subscripted terms are as shown in Figure 7. By the central difference method, the first derivative of the void ratio with respect to space is

$$\frac{\partial e}{\partial z}(z_i, t_j) \simeq \frac{1}{2\delta}(e_{i+1,j} - e_{i-1,j}) \tag{17}$$

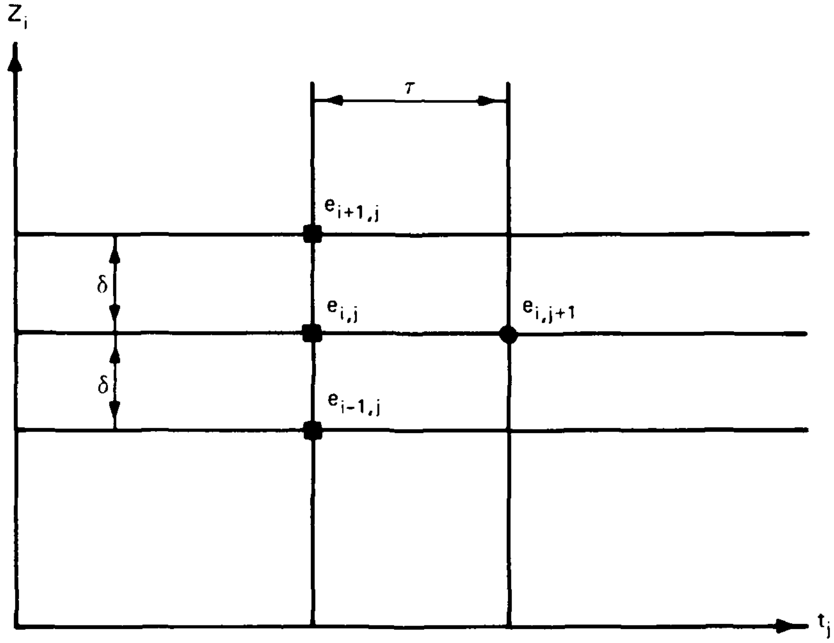

Figure 7. Finite difference mesh showing discrete increments of time and space

The second derivative of the void ratio with respect to space is

$$\frac{\partial^2 e}{\partial z^2}(z_i, t_j) \simeq \frac{1}{\delta^2}(e_{i+1,j} - 2e_{i,j} + e_{i-1,j}) \tag{18}$$

where the terms are as shown in Figure 7. These derivatives are used to convert the governing differential equation into the difference equation, which can then be coded for computer solution.

Difference equation

86. To simplify computations, the governing equation may be rewritten in the form

$$\left\{\gamma_c \beta(e) + \frac{\partial}{\partial z}\left[\alpha(e)\right]\right\} \frac{\partial e}{\partial z} + \alpha(e)\,\frac{\partial^2 e}{\partial z^2} + \gamma_w \frac{\partial e}{\partial t} = 0 \tag{19}$$

where

$$\gamma_c = \gamma_s - \gamma_w \tag{20}$$

$$\beta(e) = \frac{d}{de}\left[\frac{k(e)}{1 + e}\right] \tag{21}$$

$$\alpha(e) = \frac{k(e)}{1 + e}\,\frac{d\sigma'}{de} \tag{22}$$

Substituting the relationships from Equations 16-18 into Equation 19 yields the governing equation in finite difference form:

$$e_{i,j+1} = e_{i,j} - \frac{\tau}{\gamma_w}\left[\left\{\gamma_c\ \beta(e_{i,j}) + \left[\frac{\alpha(e_{i+1,j}) - \alpha(e_{i-1,j})}{2\delta}\right]\right\}\left(\frac{e_{i+1,j} - e_{i-1,j}}{2\delta}\right) + \alpha(e_{i,j})\left(\frac{e_{i+1,j} - 2e_{i,j} + e_{i-1,j}}{\delta^2}\right)\right] \tag{23}$$

Simulation of nonlinearity

87. The nonlinearity of the equation may be simulated by recalculating the functions $\alpha(e)$ and $\beta(e)$ for the current void ratio at each point in the space grid; this calculation is repeated at each time step.

88. The nonlinearity of the material properties is accounted for in the following manner. The $e - \sigma'$ and $e - k$ relationship laboratory data points are analyzed to determine best-fit equations of the form $y = ax^b + c$.

From these equations, values of $\alpha(e)$ and $\beta(e)$ at any needed void ratio can be calculated.

Calculation of settlement

89. To calculate the settlement at any given point in a compressible layer, the difference in the present height at that point and the initial height at the same point must be calculated. In terms of the coordinate systems used in this document, settlement (S) of a point is determined by subtracting its convective, ξ , coordinate from its Lagrangian, $\bar{a}$, coordinate such that

$$S(z,t) = \bar{a}(z,0) - \xi(z,t) \tag{24}$$

(Poindexter 1988) or settlement may be determined by integration of Equations 10 and 11:

$$S(z,t) = \int_0^z [1 + e(z,0)]\, dz - \int_0^z [1 + e(z,t)]\, dz \tag{25}$$

The numerical integration of Equation 25 can be accomplished by use of Simpson's Rule since data are generated at each finite difference mesh point during solution of the governing consolidation equation.

90. The degree of consolidation at time t, U_t , is defined in this calculation procedure as the ratio of current settlement to final settlement of the entire layer,

$$U_t = \frac{S(\ell,t)}{S(\ell,\infty)} \tag{26}$$

where $S(\ell,\infty)$ is the ultimate settlement of the layer when all excess pore water pressure has dissipated. It should be noted that this procedure for determining degree of consolidation in finite strain theory differs from the method commonly used for small strain theory where the percentage of excess pore water pressure dissipated is used to calculate degree of consolidation.

Stresses and pore water pressures

91. After determination of the void ratio throughout the layer, the void ratio-effective stress relationship can be used to obtain the effective stress distribution in that layer. The static pore water pressure can also be determined by multiplying the unit weight of water by the height of the free water surface above the point of interest, as expressed by

$$u_o(z,t) = \gamma_w[h_1 - \xi(z,t)] \tag{27}$$

where

h_1 = height of free water surface above data plane, $z = 0$

ξ = convective coordinate of mesh point at time in question

92. The total stress in the compressible layer can be calculated by summing the total weights of solids and fluids in a unit area above the point of interest, as follows:

$$\sigma(z,t) = \gamma_w \left[h_2 + \int_z^{\ell} e(z,t)\, dz \right] + \gamma_s \int_z^{\ell} dz \tag{28}$$

where

h_2 = height of free water surface above the top ($z = \ell$) of the compressible layer

ℓ = layer height in material, z , coordinates

93. Since both the total and effective stresses have been determined, the effective stress principle can be used to calculate total pore water pressure:

$$u_w(z,t) = \sigma(z,t) - \sigma'(z,t) \tag{29}$$

Using the values of total and static pore water pressure, the excess pore water pressure can then be calculated by the expression

$$u(z,t) = u_w(z,t) - u_o(z,t) \tag{30}$$

A computer program can be developed to calculate stresses and pore pressures by numerical integration of Equations 29 and 30 for all material nodes.

Shear Strength Estimation

94. As consolidation occurs in the dredged material deposit, pore water is extruded from the deposit, the void ratio decreases, and the effective stress increases; as a result, the shear strength of the material increases. This increase in shear strength results in a deposit that is less susceptible to slope instability and erosion. Thus, it is important to obtain information on the shear strength of the deposit both initially and as time progresses.

95. A number of methods are available to determine the shear strength of subaqueous soil deposits. These methods include in situ testing methods, laboratory testing of undisturbed soil samples, and empirical correlations between shear strength and various properties of disturbed samples.

96. Recently, an empirical correlation was developed that relates the undrained shear strength of remolded clays to their Atterberg limits and activity (Carrier and Beckman 1984). This correlation was developed using a wide range of clays with varying plasticity and liquidity indices. This relationship was intended to be applicable to phosphatic clay wastes (slimes), dredged material, and normally consolidated clays such as marine sediments. The purpose for developing this empirical correlation was to allow a priori prediction of the conditions within hydraulic fills such as dredged material disposal sites and mine tailings impoundments.

97. The Carrier and Beckman empirical correlation was based upon data from individual shear strength tests on numerous clays. Data for 8 phosphatic clays and 15 natural clays were used; all shear strength values were obtained on remolded specimens. Various testing methods were utilized to obtain the shear strength data, including unconfined compression tests, unconsolidated undrained triaxial compression tests, vane shear tests, and fall cone penetration tests (Carrier and Beckman 1984). The following equation for the undrained shear strength of clays was obtained by Carrier and Beckman:

$$S_R = P_{atm}\left\{\frac{0.166}{0.163 + \dfrac{37.1e - (PL)}{(PI)[4.14 + (act.)^{-1}]}}\right\} \qquad (31)$$

where

S_R = undrained shear strength

P_{atm} = atmospheric pressure

PL = plastic limit

PI = plasticity index

act. = activity = plasticity index divided by the percent clay in the sample

This equation relates the shear strength to the plasticity index, activity, and void ratio of the material. This relationship can be used to estimate the shear strength of a dredged material deposit as it undergoes consolidation.

98. By using Equation 31 and the vertical effective stress, a ratio of shear strength to effective stress can be obtained. This c/σ' ratio is normally designated as $c/\bar{p}n$ and is commonly referred to as the "c/p ratio" (Terzaghi and Peck 1967), where c is the undrained shear strength and σ' is equal to $\bar{p}n$, the effective overburden pressure. It should be noted that the c/p ratio is a function of the void ratio, which will change as consolidation proceeds. Thus, the c/p ratio will not be constant during consolidation. Additionally the c/p ratio is dependent upon the initial water content of the material and therefore can vary enormously for very soft, low-activity clays.

Extruded Water Volume

99. As consolidation occurs, water is extruded from the soil mass. The rate at which this water is extruded will be directly dependent upon the rate at which consolidation proceeds. The extrusion of pore water from a mound of dredged material can become extremely important when the material is contaminated with toxins, heavy metals, or organic hydrocarbons.

100. Design of a capped disposal mound is intended to isolate the contaminated dredged material from the surrounding environment. The design thickness of the cap is supposed to be great enough to (a) prevent chemical contaminant migration into the water column (chemical considerations) and (b) prevent burrowing organisms from traveling through the cap and into the contaminated material (biological considerations). Results of numerous research projects, both laboratory and field investigations (O'Connor and O'Connor 1983; Mansky 1984; Brannon et al. 1985, 1986; Gunnison et al. 1986; Environmental

Laboratory 1987), indicate that the required thickness of cap is usually controlled by the biological considerations. Required cap thickness is typically 1 to 3 ft (Brannon et al. 1985, 1986; Gunnison et al. 1986; Environmental Laboratory 1987).

101. The depth of capping material required and placed as a result of the chemical and biological testing has been found to maintain chemical migration into the water column within acceptable levels. This indicates that either the cap has sufficient volume of voids to "store" all contaminated pore water extruded from the contaminated material or, more likely, the combination of cap storage volume, contaminant sorption to cap particles, and rate of dredged material consolidation is such that the release of contaminants from the cap surface is slow enough to maintain contaminants at acceptable levels.

102. For purposes of this evaluation, the volume of water extruded from the dredged material over time will be considered to be equal to the consolidation settlement of the dredged material. This approach will be sufficiently accurate for this analysis since the material will be saturated, and therefore any change in height can be attributed to a change in water content.

Computerized Solution

103. The solution of the governing equations using the techniques presented here is incorporated into the computer program MOUND. A user's manual giving specifics of program organization, input requirements, program listing, output format, and other information necessary for program use in predicting settlements at actual disposal sites is being prepared for publication as a separate WES technical report.

104. The program MOUND is an extensively revised and expanded version of the computer program PCDDF (Cargill 1985). This program incorporates an explicit finite difference procedure to solve self-weight consolidation problems by utilizing the finite strain theory of consolidation. The program can calculate the consolidation not only of the dredged material mound but also of compressible foundation soils. Drainage boundaries in this program may be permeable, semipermeable, or impermeable.

105. This program is intended to be used as an aid in determining site capacity of subaqueous dredged material disposal areas where settlement of the mound will occur as a result of self-weight consolidation of the mounded

material and/or the surcharge caused by the capping material. The program permits placement of subsequent deposits of dredged material at any time period during the analysis, although the same properties must be used for all deposits of dredged material. The surcharge loading of any capping material can be applied to the dredged material surface. One compressible foundation soil layer may be analyzed by this program, and its compressibility can be different from that of the dredged material.

106. Another feature of MOUND is the calculation of soil stresses and pore pressures during the consolidation process. These values are helpful in assessing soil strength. The recently developed Carrier and Beckman (1984) empirical procedure for estimating shear strength has been incorporated into the computer program.

PART V: FIELD SITE CONDITIONS

107. Several field projects have been conducted or are being planned for the near future to demonstrate the engineering aspects of subaqueous disposal of contaminated dredged material. Conditions at the sites and the methods of monitoring the postdisposal behavior of the deposited material vary from site to site.

108. The sites investigated in this study are located within two Corps of Engineers Divisions--the North Pacific Division and the New England Division. The field site within the North Pacific Division, Seattle District, is located in the Duwamish Waterway. In the New England Division, study sites are located at the Central Long Island Sound disposal site; the three mounds investigated here are described as the Stamford-New Haven North, Stamford-New Haven South, and the Field Verification Program (FVP) mounds. Contained in the following paragraphs are descriptions of each of these field sites, including the disposal and monitoring activities at each. The locations of these sites, as well as several proposed sites, are shown in Figure 8. All of the sites are located in industrialized and heavily populated regions of the northern United States.

Duwamish Waterway Site

109. A field demonstration project was conducted on the West Coast of the United States to evaluate various aspects of contained aquatic disposal of contaminated dredged material. This 18-month study was conducted in the Duwamish Waterway, a heavily industrialized river system located in Seattle, WA. The Duwamish River, shown in Figure 9, divides to form the East Waterway and the West Waterway as it approaches Elliott Bay. Also shown in this figure are the locations of the contaminated shoal that was dredged, the source of capping material, and the capping test site. Both the contaminated shoal and the test site are located within the tidal influence of the Pacific Ocean. The tidal range in this area is approximately 11 ft.

Background

110. The investigations conducted at the Duwamish Waterway site were a joint effort between the Seattle District and the Waterways Experiment Station. The Seattle District was responsible for the planning and operational

Figure 8. Location of existing and proposed subaqueous disposal sites

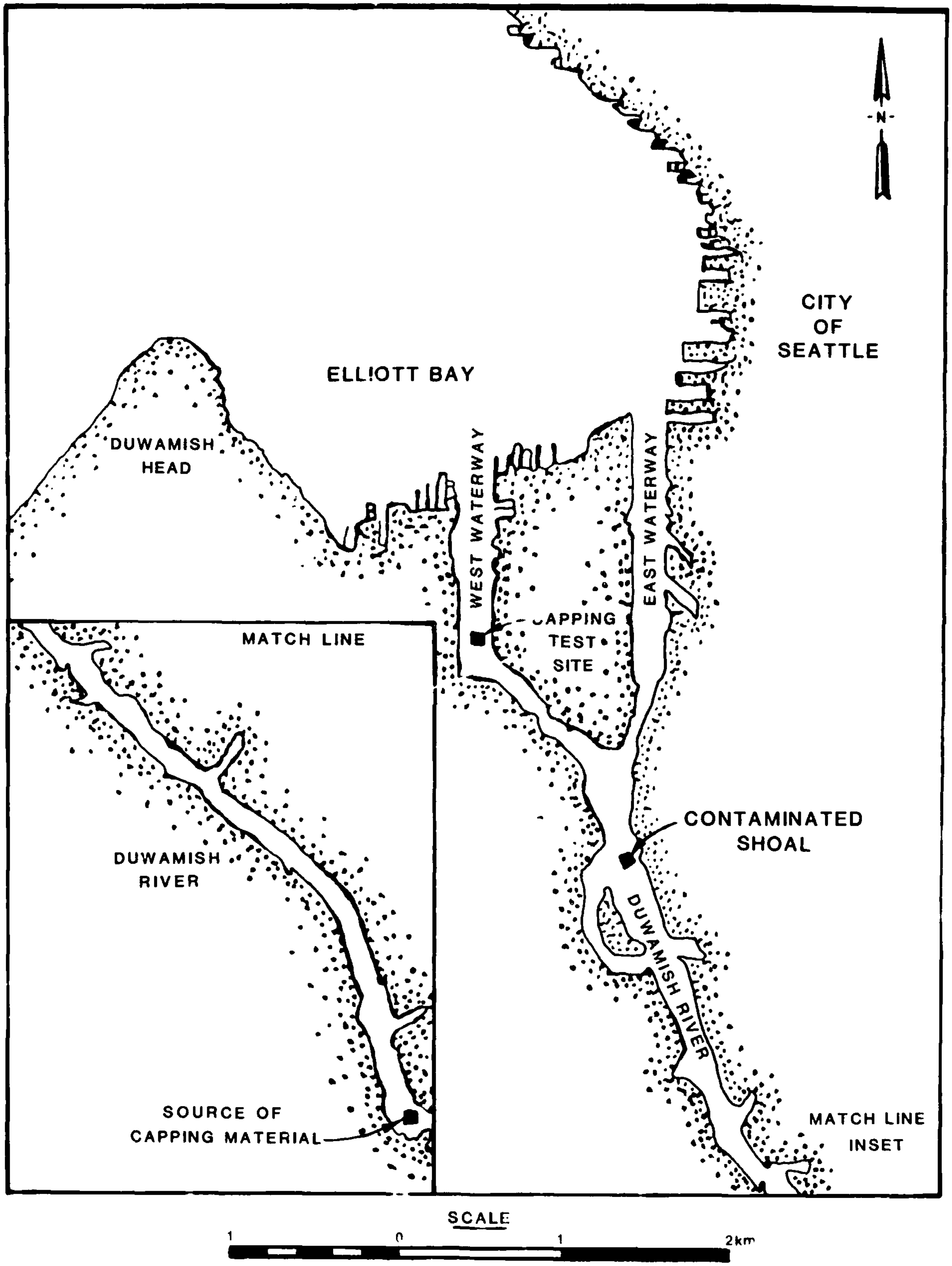

Figure 9. Vicinity map for Duwamish Waterway disposal site (after Sumeri 1984)

aspects of the dredging and disposal operations (Sumeri 1984), and WES participated by conducting the 18-month monitoring and evaluation program. Initial results of this study were reported by Truitt (1986).

111. In the past, significant restrictions had been placed on disposal of dredged material from the lower reaches of the Duwamish Waterway because of the contaminated nature of the sediments. A lack of acceptable, economical disposal sites caused dredging of these reaches to be minimized in recent years. When shoaling had progressed to the point of interfering significantly with shipping, it was decided to dredge the contaminated sediments and place them in a CAD site. A small demonstration project was planned to allow evaluation of the proposed dredging and disposal techniques (Sumeri 1984).

112. The shoal to be dredged for this demonstration project was small, containing only 1,100 cu yd of contaminated material. Because a number of contaminants were present in sufficient amounts to preclude open-water disposal at a designated Elliott Bay disposal site, an alternate site had to be located. Review of previous bathemetric surveys indicated that a series of depressions existed in the West Waterway. The southernmost depression was selected as the disposal site. It is located approximately 4,000 ft up the waterway, south of its juncture with Elliott Bay. The disposal site is nominally 100 to 150 ft wide, 300 ft long, and 6 ft deep.

Disposal of material

113. Dredging of the contaminated shoal occurred on 26 March 1984. The sediment was removed from the river bottom by clamshell dredge and was placed in a split-hulled, bottom-dumping barge. Approximately 1,100 cu yd of material was removed from the contaminated shoal. Removal of this quantity of material was sufficient to provide the necessary navigation depth in the channel, but was also small enough to be transported in one barge. Thus, the disposal operation could be limited to one discrete discharge of contaminated material.

114. Disposal of the contaminated material occurred near higher low tide on the morning of 27 March. This was the higher of the day's two low tides and produced a water surface elevation that was approximately 6 ft above datum (mllw); this resulted in a maximum depth of water at the disposal site of 70 ft. Currents in the vicinity were expected to be weak and variable for at least 2 to 3 hr.

115. The split-hulled barge was moved into position by tugboats. Precise positioning was accomplished using surveying equipment stationed on land along the waterway (Sumeri 1984). When the correct position was obtained, the barge was opened. The mass of contaminated material exited the opened barge in approximately 19 sec, and its descent was traced by side scan sonar. The material moved rapidly to the bottom as a well-defined mass.

116. The capping operation was accomplished over a period of 3 days, beginning on 28 March. During the course of the capping operation, a total of approximately 4,000 cu yd of clean sand was placed over the mound of contaminated material. Each of three barge loads of sand was gently discharged at the disposal site during periods of low tide and thus low current velocities. For each disposal operation, the barge hull was slowly opened over a period of 45 to 60 min. This allowed the sand to be "sprinkled" over the contaminated mound at a controlled rate. By so placing the capping material, it was hoped that displacement of the soft contaminated material would be minimized and cap coverage of the site would be maximized. Hydrographic surveys were conducted at the disposal site after each capping operation to verify results and to determine subsequent positioning requirements.

Sampling and monitoring activities

117. Because of the interest in this capping demonstration project, a considerable amount of sampling and monitoring effort was expended before, during, and after all three phases of operations (dredging, disposal, and capping). Discrete water column samples were taken throughout the operations at both the dredging and disposal sites; these samples were analyzed for chemical constituents and total suspended solids. Additionally, nephelometry equipment provided continuous monitoring of turbidity levels. Temperature, salinity, and current measurements were also made periodically from the various sampling boats. Side scan sonar was used to provide images of (a) the bottom of the shoal area both before and after dredging, (b) the descent of the contaminated material from the barge to the disposal site, and (c) the turbidity plumes caused during dredging and disposal operations. The abovementioned sampling and monitoring is relevant to water quality analyses and is reported in detail by Truitt (1986). The sampling and monitoring activities of interest in this document are discussed in the following paragraphs. A more detailed discussion is provided by Poindexter (1988).

118. Extensive field investigations and sample collection efforts were expended at the Duwamish CAD demonstration site to characterize the postdisposal behavior of the dredged material deposit. To obtain these data, various methods were employed, including hydrographic surveys, sediment sampling, and settlement plates.

119. Hydrographic surveys were conducted by Seattle District personnel throughout the project. Bottom profiles of the disposal site were obtained on 25-ft centers across the region in which the disposal site was located. Disposal site profiles were obtained both before and after disposal of the contaminated material. Additional profiles were provided after placement of each barge of capping material. These surveys were used to determine the thickness and location of material deposited during various disposal operations. Subsequent hydrographic surveys have been conducted to monitor any changes in elevation of the mound.

120. Considerable sediment sampling was conducted during the Duwamish field demonstration project. Soil borings were taken at the contaminated shoal and at the disposal site prior to dredging; additional borings were made at the disposal site after placement of the contaminated dredged material and then periodically after placement of the capping material. During the dredging process, representative composite (grab) samples were taken of both the dredged material and the capping material as these sediments were being placed into transport barges.

121. To supplement information obtained from the hydrographic surveys and Vibracore borings, settlement plates were installed at the disposal site. Because of the sequenced disposal, a series of multitiered or telescoping settlement plates (Figure 10) were designed and fabricated for use at the Duwamish Waterway disposal site (Poindexter 1984). Use of settlement plates of this design allowed changes in thickness of the various layers of material to be separately monitored. For example, consolidation settlement of the contaminated dredged material could be delineated from any potential erosion of the capping material. (Because the cap was composed of sandy material, it was not expected to undergo any consolidation.) If the elevation of the settlement plate risers had been tied into a known elevation outside the disposal area, any settlement of the foundation soils could also have been monitored.

122. The telescoping arrangement permitted placement of the lower plates on the foundation soil prior to disposal of the contaminated material and

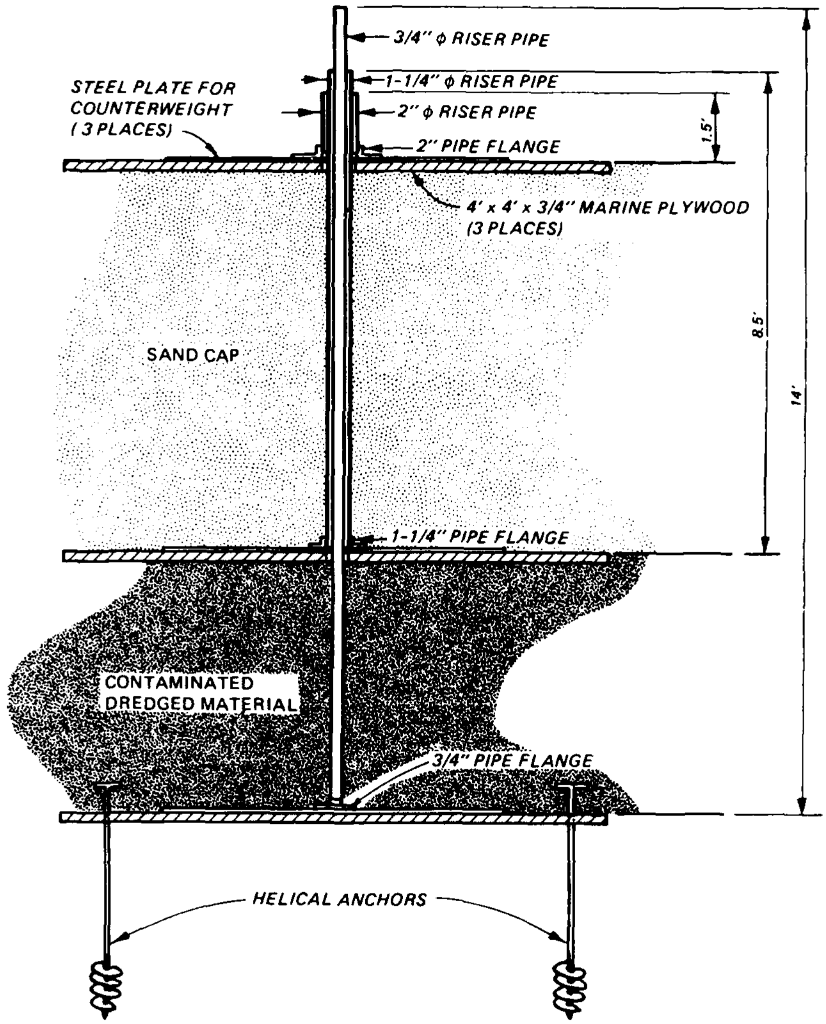

Figure 10. Multitiered settlement plates used to measure settlement of various layers of material

placement of the second and third tiers of settlement plates after disposal of, respectively, the dredged material and the capping material. Each bottom plate was anchored to the foundation soil by two 4-ft helical earth anchors. Despite the anchoring arrangement, several of the plates were overturned or moved laterally by the impact of the dredged material. The unanticipated presence of approximately 2.25 ft of very soft organic silts and clays overlying the firmer foundation soils is believed to be a major factor in the instability of the plates. Readings on the remaining usable settlement plates were taken after each stage of disposal and at 2 weeks, 6 months, and 18 months after disposal operations were completed. Divers were required for both installation and reading of the settlement plates. Use of geologically

or geotechnically trained divers would have provided additional useful information regarding disposal site conditions and might have allowed modification of settlement plate installation to preclude loss of the plates.

Site conditions

123. From the Vibracore samples taken at the disposal site, the stratigraphy of the foundation soils at the site was determined. The foundation soil upon which the mound of dredged material was placed included approximately 2.25 ft of soft material that was composed of sandy clays and organic or clayey silts. This soft soil was underlain by at least 15 ft of medium dense sandy silt and silty sand.

124. The presence of the soft soils above the firmer silts and sands had not been detected in preliminary site investigations conducted by the Seattle District, and was not anticipated during planning of the monitoring efforts. Thus, although the settlement plates were designed with 4-ft-long anchors, they were located above 2.25 ft of soft material and were not as stable as they should have been. As a result, it is not surprising that some of the settlement plates were overturned or displaced laterally during disposal of the dredged material.

125. Placement of the contaminated dredged material resulted in formation of a mound in the depression of the disposal site. This mound was just over 3 ft high at its thickest point and generally had a height of 24 to 36 in. The depth of material at the settlement plates was measured to be 30 in. Thus the average initial thickness of the mound was assumed to be 30 in.

126. Application of the 4,000 cu yd of sandy capping material resulted in uniform coverage of the contaminated dredged material. The cap was typically 1 to 2 ft thick across most of the disposal area; a thickness of at least 3 ft was maintained over the central portion of the mound.

127. The soil profile that will be used in subsequent analyses is shown in Figure 11. It consists of a conical mound of contaminated dredged material with a maximum thickness of 3 ft; this material is covered by a sand cap with a center thickness of 3 ft. This mound of material is located on foundation soils consisting of incompressible silts and sands that extend to a depth of at least 15 ft.

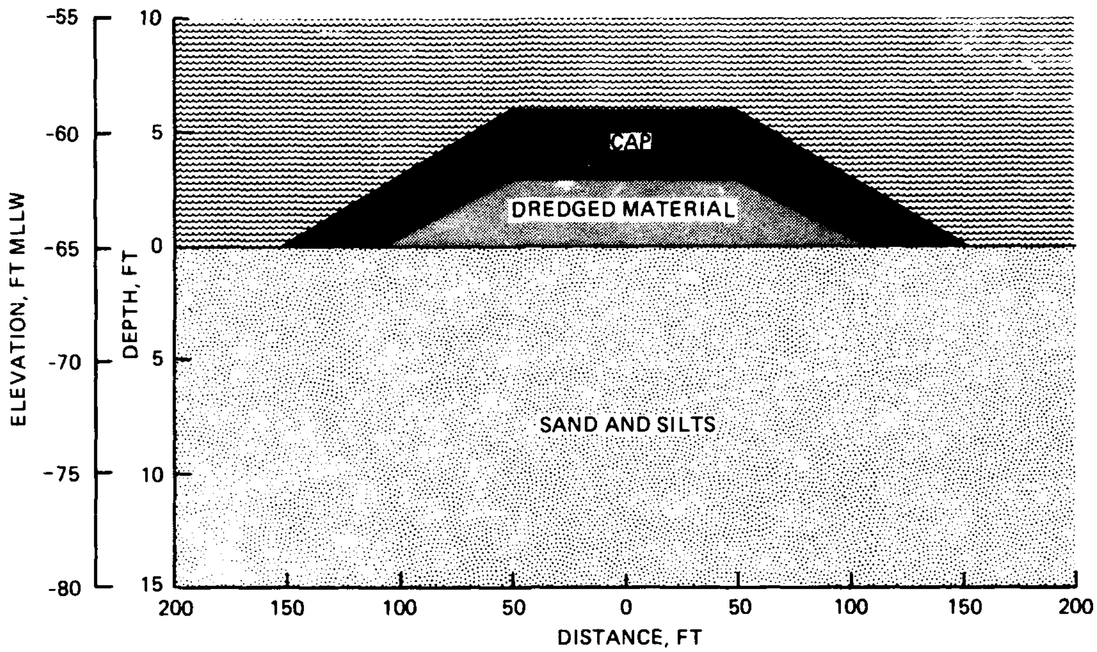

Figure 11. Idealized profile of Duwamish mound and foundation soils

Long Island Sound Sites

128. Long Island Sound is an estuarine water body located between Long Island, New York, and the State of Connecticut. It is a long, narrow body of water, connected with the Atlantic Ocean on its eastern end and with the East River and New York Harbor on its western end. Several designated disposal sites are located within the sound, one of which is the Central Long Island Sound (CLIS) disposal site.

129. The CLIS site lies 7 miles south of New Haven, CT, in the central portion of the Sound, as shown in Figure 12. This disposal site is a rectangular area of approximately 2 square miles. The bottom surface is relatively level and slopes slightly to the south. The water depth at the disposal site is approximately 65 ft. Currents are tidal induced and have a maximum value of 1.5 fps. The foundation soil profile at the CLIS disposal site consists of approximately 33 ft of marine silt overlying 82 ft of sands and gravels. Below the sand and gravel is approximately 200 ft of freshwater lake clays, which are underlain by 33 ft of till or cretaceous sand and gravel and Paleozoic bedrock.

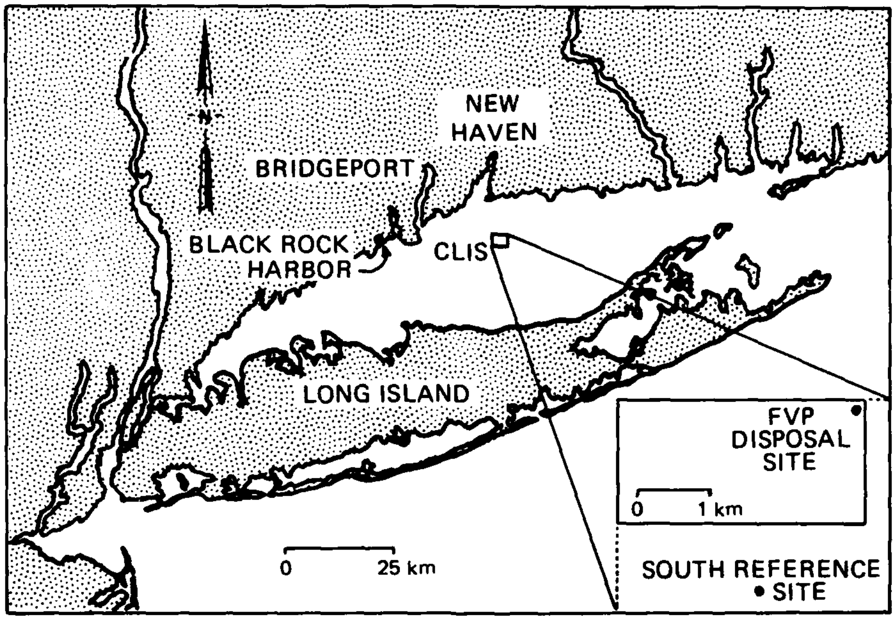

Figure 12. Location of Central Long Island Sound disposal site

130. Three mounds of interest are located in the CLIS disposal site: the FVP mound, the Stamford-New Haven North (STNH-N) mound, and the Stamford-New Haven South (STNH-S) mound. Figure 13 shows the relative locations of these mounds.

131. Monitoring of the dredged material settlement over time at each of these mounds has consisted principally of hydrographic surveying (Morton 1980, 1983). The area containing each mound was surveyed prior to mound formation, during and immediately after formation, and periodically since that time. The precision bathemetric surveys were conducted along an established survey grid in order to provide replicate surveys. The grid lines were oriented in an east-west direction and were spaced at 82-ft intervals. A computerized microwave positioning system was used to provide survey control.

132. Although precision bathymetric surveys were conducted, there are technical limitations in any typical oceanographic remote surveying technique. Problems associated with navigational accuracy and resolution of the echo soundings, in conjunction with a nonplanar seafloor, can create variations in results which may be insignificant for most monitoring activities, but which may become significant when comparing predicted mound settlement with field

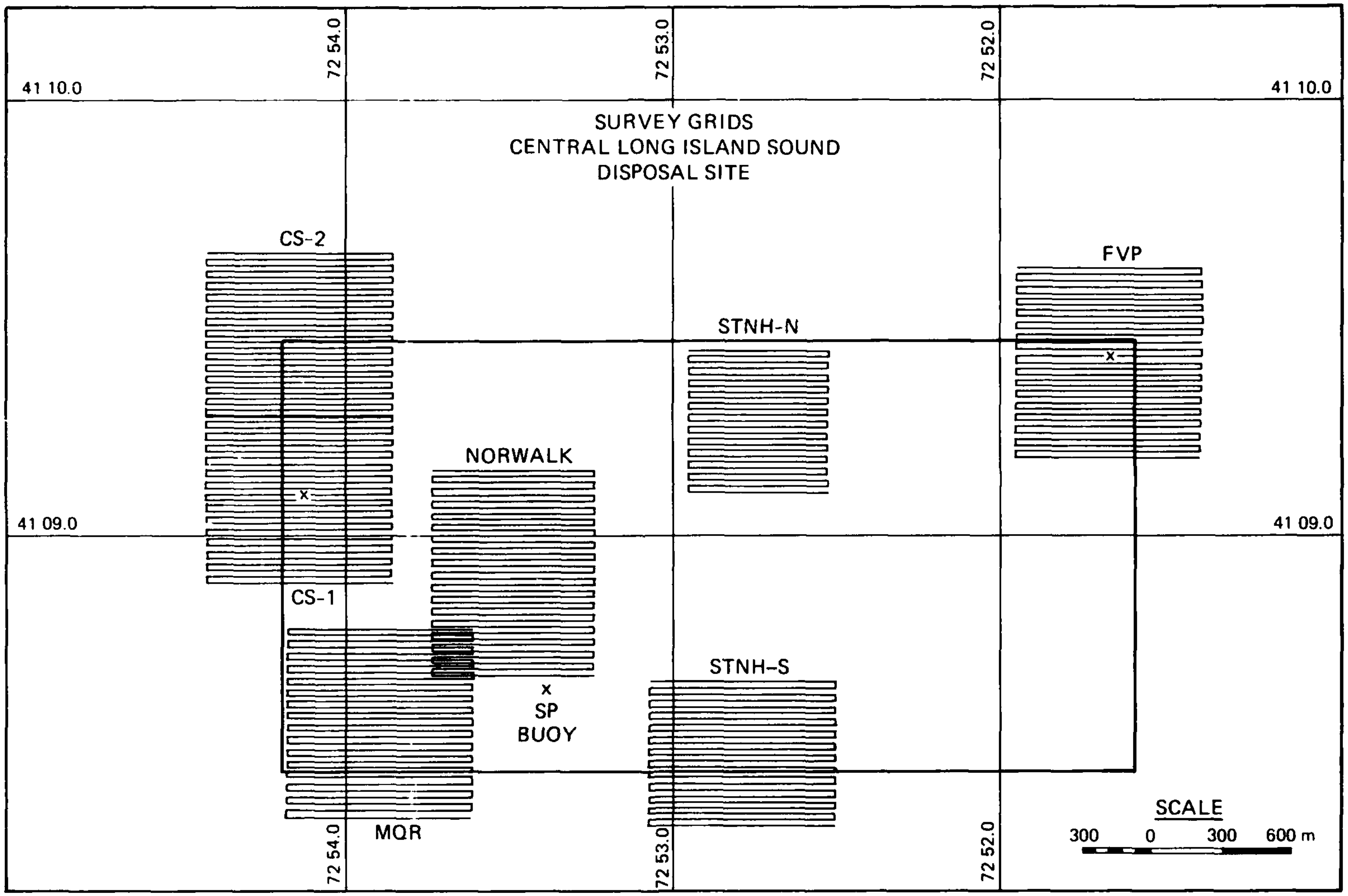

Figure 13. Location of FVP, STNH-N, and STNH-S mounds within the CLIS disposal site

observations. Thus, precision of the survey results available for the CLIS sites was evaluated.

133. Analysis of the surveys conducted for the FVP, STNH-N, and STNH-S mounds indicated that a survey precision of ±0.7 ft should be expected in these data. This value was determined by comparing the elevations at specific points within the survey grid, but outside the mound, for the various survey dates. A typical example of the variation found in mound baseline data is shown in Figure 14.

134. It should also be noted that the only data available in this study were computer-generated plots which are subject to a human error during data entry. This error is illustrated in Figure 14 on the lane 14 survey where a large triangular protrusion is shown as the survey lines begin to ascend up the mound side slope.

135. Excepting human-introduced errors due to plotting or reading errors, the absolute technique precision in the resurvey data is ±0.7 ft. The primary cause of this is the inability to control the position of the survey vessel relative to the grid line. Note that the survey data in Figure 14 are referenced to lanes not lines, indicating that the authors were aware of this technique problem (Morton 1983).

Field Verification Program Mound

Background

136. Section 103 of the Ocean Dumping Act and Section 404 of the Clean Water Act require that field-verified, state-of-the-art procedures for prediction of environmental effects be used to evaluate any proposed dredged material discharges addressed by these Acts. Specific aspects of dredged material disposal, such as potential bioaccumulation and biomagnification of contaminants in aquatic organisms, as well as degradation of water quality, must be considered in these evaluations. The overall impact of all disposal alternatives must also be considered.

137. The Field Verification Program was a cooperative effort between the Corps of Engineers and the USEPA to field verify dredged material testing procedures for predicting the impact of aquatic disposal, upland disposal, and wetland creation, as required under Sections 103 and 404. Through the FVP, promising test procedures developed by both the Corps and the USEPA were

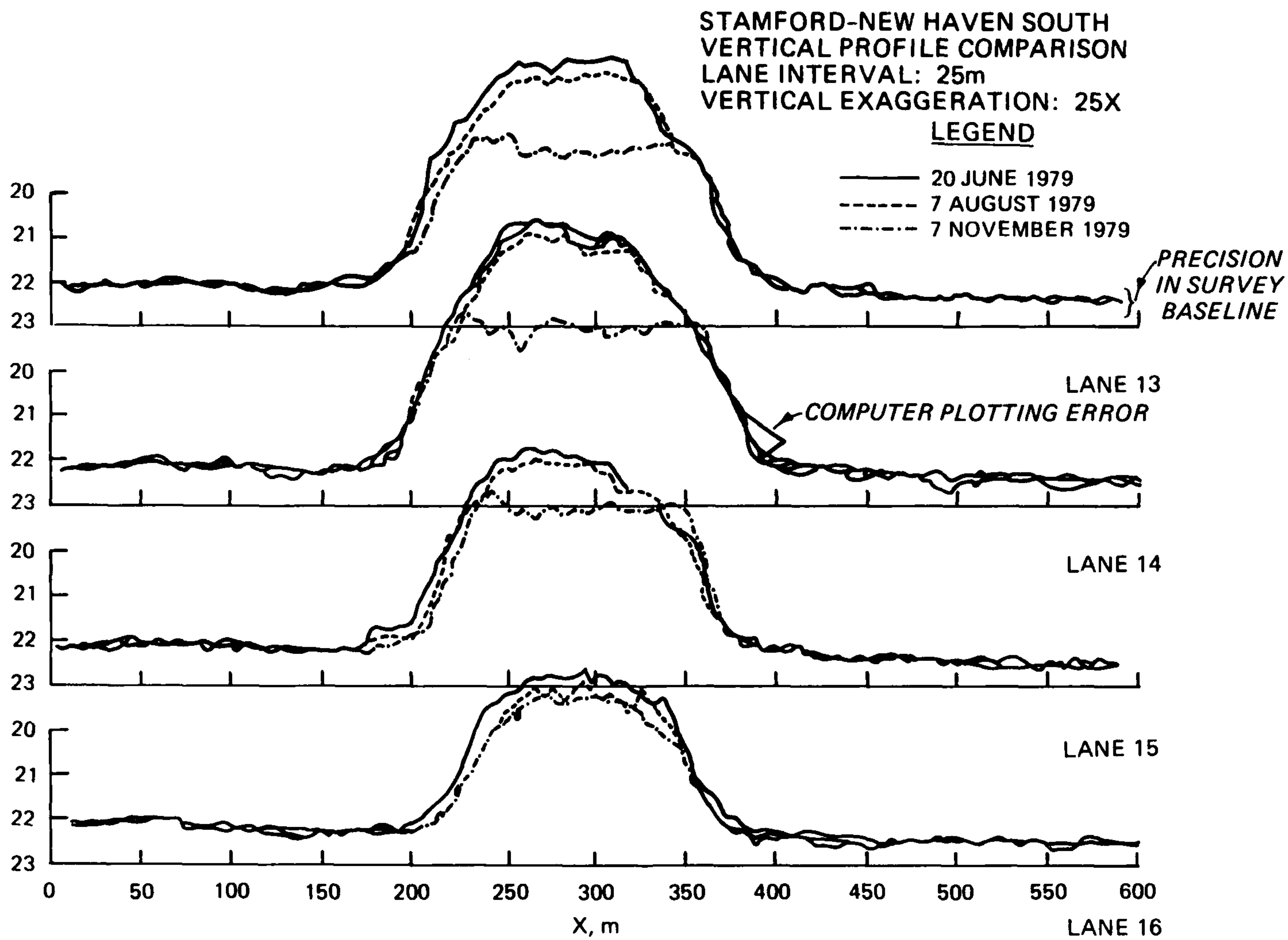

Figure 14. Typical hydrographic survey data indicating survey precision (after Morton 1983)

applied to a dredging project in the New England Division. Dredged material from a single maintenance dredging project at Black Rock Harbor (BRH) in Bridgeport, CT (see Figure 15), was placed in an aquatic site, a confined upland site, and a confined wetland site. The FVP upland/wetland studies were conducted at Tongues Point, CT, adjacent to Bridgeport Harbor, while the aquatic studies, which are of interest in this investigation, were conducted at the CLIS aquatic disposal site. These field projects provided an unusual opportunity for direct comparison of the environmental consequences resulting from disposal of the same material in various disposal environments. The FVP results will provide guidance to the Corps and USEPA field elements for assessing the potential impact of disposal alternatives.

Disposal of Material

138. Material was dredged from BRH in the spring of 1983 for placement at the CLIS disposal site. The material was dredged by clamshell and placed into bottom-dump scows for transport to the disposal site. The scows were individually moved to the designated disposal location. When accurately positioned adjacent to a taut-wire moored buoy designating the disposal site, the scows

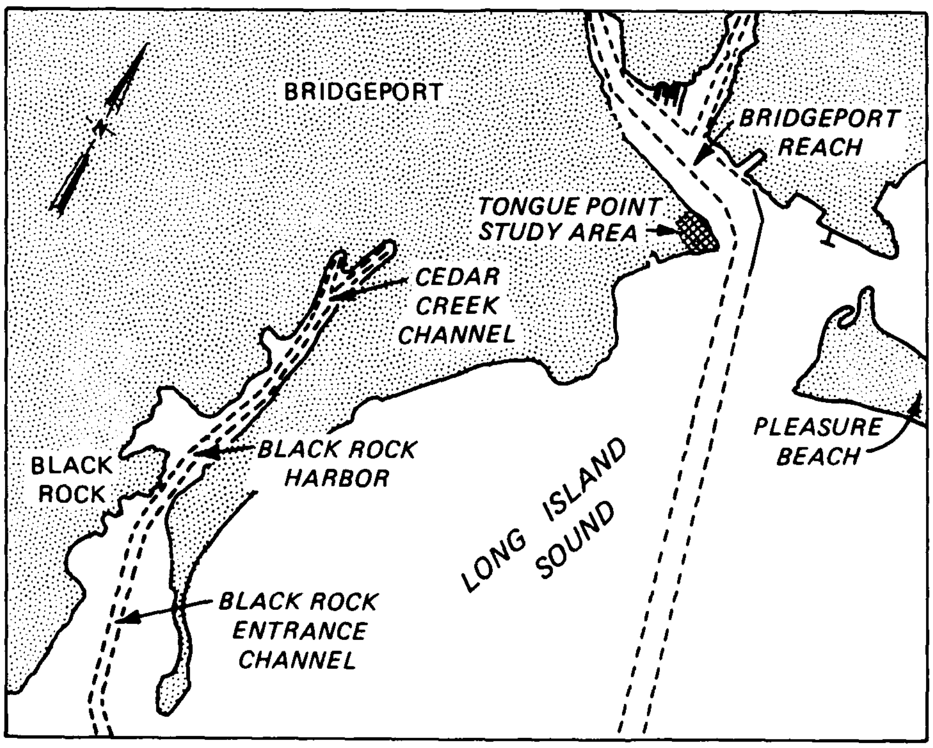

Figure 15. Location of Black Rock Harbor dredging project and Tongues Point upland/wetland disposal areas

were opened, and the dredged maerial was discharge. Disposal of the BRH sediment was accomplished during the period 28 April 1983 to 5 May 1983, with a total of 72,000 cu yd of material being placed to form the FVP mound. This mound had a height of 6.5 ft and a diameter of 660 ft (Morton 1983). No capping material was placed on this mound of dredged material.

Sampling and monitoring activities

139. Monitoring of the FVP mound has consisted principally of hydrographic surveying of the mound and surrounding area. Precision hydrographic surveys have been periodically conducted, with the initial survey being conducted before disposal of the dredged material. The surveys were conducted on an established survey grid in order to provide replicate surveys of the mound. The grid lines were oriented in an east-west direction and were spaced at 27.5-ft intervals. A computerized microwave positioning system was used to provide survey control. Surveys were conducted prior to disposal of the BRH sediment, immediately after disposal, and at various time intervals since the disposal operation. These surveys were used to construct a contour map of the mound and surrounding area immediately after disposal, as shown in Figure 16. Profiles of the mound were also constructed and are shown in Figure 17.

140. Additional monitoring has been conducted at the FVP site for various purposes. Numerous water quality and biological monitoring techniques have been employed to determine the environmental effects (both chemical and biological) of contaminated dredged material disposal. Results of the monitoring efforts provided data for comparison with Corps- and USEPA-predicted environmental effects.

Site conditions

141. The mound of BRH sediment at the FVP disposal location (center height 6.5 ft, diameter 660 ft) was placed on approximately 33 ft of marine silt that was overlying 82 ft of sands and gravels. The soil profile that will be used in the subsequent analyses is presented in Figure 18.

Stamford-New Haven North Mound

142. The Stamford-New Haven North mound is located at the CLIS disposal site (Figure 13). It therefore has the same general disposal site conditions and foundation stratification as the FVP mound.

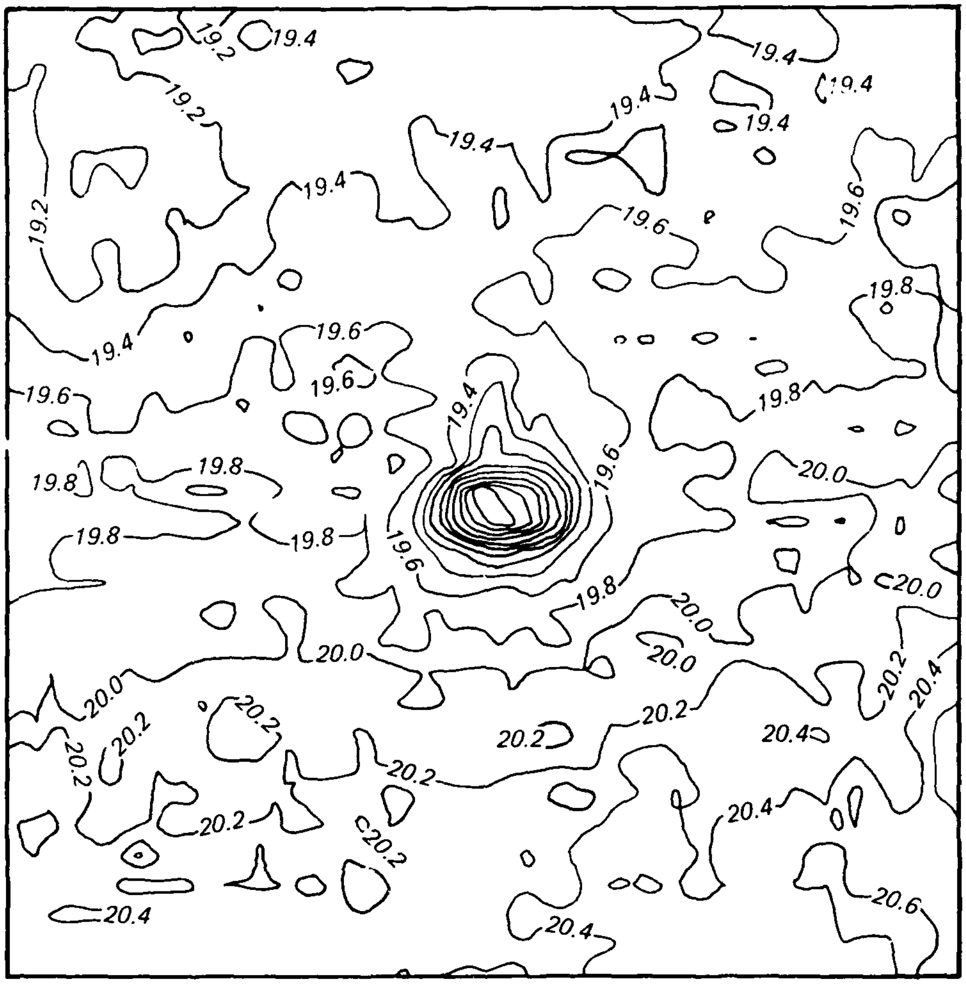

Figure 16. Contour map of FVP mound
(contour interval = 0.2 m)

Background

143. Many of the larger harbors along the coastline of Connecticut require periodic dredging in order to maintain navigable depths. Because of the heavy industrialization, many of the sediments dredged are contaminated with industrial waste products; typically found in these sediments are high concentrations of polychlorinated biphenyls and heavy metals. A large portion of this material is placed in aquatic disposal sites on the continental shelf because (a) there is a lack of suitable upland and/or intertidal disposal sites and (b) costs for transport of the material beyond the continental shelf are prohibitive. As environmental awareness and legislation have increased, the practice has evolved such that the most contaminated portions of a reach are

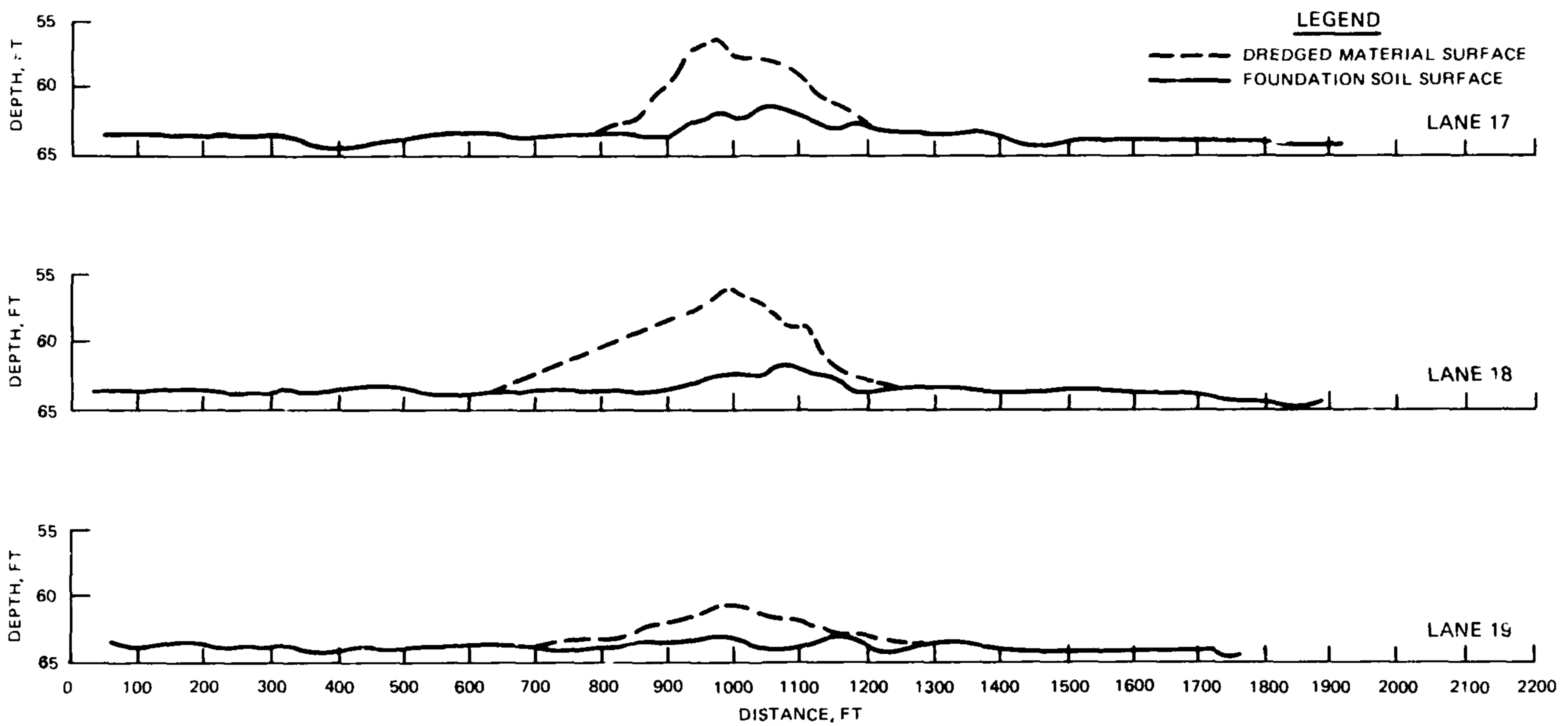

Figure 17. Typical profiles of FVP mound

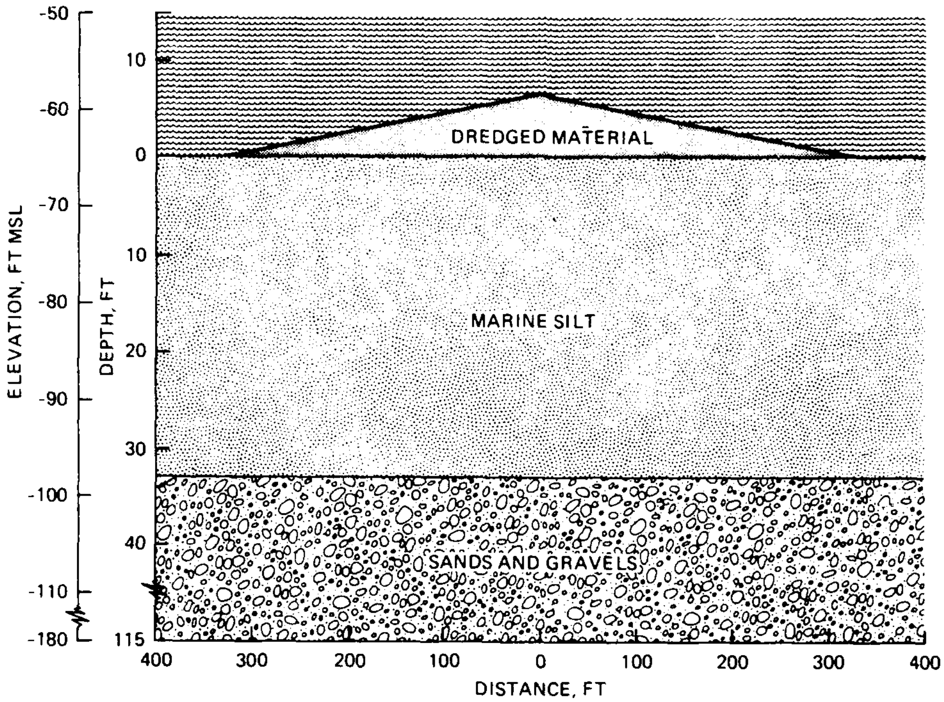

Figure 18. Idealized profile of FVP mound and foundation soils

dredged during the initial phases of a project; this material is covered with the least contaminated sediments, which were dredged during the final phases of the operation. This technique developed into larger scale capping operations in which larger volumes of contaminated dredged material are placed at designated locations and are then covered with clean, uncontaminated dredged material. One of the first and largest of such capping operations was conducted by the New England Division using dredged material from the harbors of Stamford and New Haven, CT.

144. Extreme shoaling conditions existed at both the Stamford and New Haven harbors. It was determined that these harbors would require dredging during 1979 to ensure continued passage of commercial vessels to terminals in these cities; of particular concern was access for oil-related traffic. Analyses of the sediments to be dredged indicated that the sediment in Stamford Harbor had high concentrations of heavy metals, while the New Haven material was much cleaner. Therefore, it was decided to place the contaminated Stamford material at a disposal site and cap it with the cleaner New Haven material. The New England Division then developed an elaborate

disposal and monitoring scheme to evaluate the effectiveness of the disposal operations as well as the relative abilities of different capping materials to cover and isolate the contaminated sediment. It was decided that Stamford contaminated sediment would be placed in two locations; one mound of Stamford material would be capped with silt from New Haven, and one Stamford mound would be capped with New Haven sand.

145. The Stamford-New Haven North mound is the Stamford mound that was capped with the New Haven sand. It is located in the CLIS disposal site and is 1,100 ft north of a landmark mound created with New Haven material in 1974 (see Figure 13). The STNH-N mound was located north of the old mound because tidal flow in the CLIS site is generally east-west, and thus, effects from the old mound would be minimized at this site.

Disposal of material

146. Stamford material for creation of the STNH-N mound was dredged by clamshell during the period April through June 1979. The contaminated material was placed in bottom-dump scows for transport to the disposal site. As each scow was filled, it was moved to the disposal site for discharge of the dredged material. The scows were accurately positioned over the disposal site, opened, and the dredged material discharged. The disposal operations at the STNH-N mound location occurred between 23 April 1979 and 15 June 1979 and resulted in placement of 34,000 cu yd of contaminated Stamford dredged material. On 15 June 1979, dredging and disposal of the fine-grained contaminated material were halted to avoid biological impact to oyster larvae. At this time, placement of the capping sand from New Haven began.

147. The sandy capping material was dredged between 15 and 21 June. The hopper dredge *Essayons* was used to remove the sand from the mouth of New Haven harbor and to transport and dispose of it at the STNH-N mound. Approximately 43,100 cu yd of sand was placed to cap the STNH-N mound.

Sampling and monitoring activities

148. Sampling and monitoring at the STNH-N mound has been conducted by NED as a part of the Disposal Area Monitoring System (DAMOS). Before dredging began, physical, chemical, and biological monitoring programs commenced at the STNH-N location. This monitoring included precision bathymetric mapping of the site, visual observations by divers, sample collection for chemical analyses, and sampling of benthic populations for recolonization and bioaccumulation studies. During the disposal operations, bathymetric surveys, diver

observations, and chemical sampling were used to monitor the location/distribution of the contaminated material and to manage the capping operation to ensure adequate coverage of the contaminated material. Additionally, this monitoring provided depths or thicknesses of both the dredged material and the capping material. Subsequent to disposal, replicate bathymetric surveys have been and continue to be periodically conducted to monitor the mound. A contour map (Figure 19) and profiles (Figure 20) were constructed from results of the bathymetric surveys. It should be noted that diver observations indicated that the sand cap provided a smooth surface on the mound.

149. In addition to the DAMOS monitoring program, samples were collected from the STNH-N mound during September 1986. These samples were collected for engineering classification and consolidation testing by WES. Other samples from the mound had been collected and tested previously by other institutions, but it was deemed prudent to obtain samples for additional testing using WES geotechnical laboratory testing procedures.

Site conditions

150. The Stamford-New Haven North mound was constructed of contaminated dredged material from Stamford harbor. This material had an initial maximum height of 6.9 ft. It was capped with New Haven sand, which had a thickness of 4.9 ft. The STNH-N mound had a diameter of 1,000 ft. This mound was placed on a foundation consisting of 33 ft of marine silt and 82 ft of sands and gravels. Figure 21 shows the mound and foundation soil profile that will be used in subsequent analyses.

Stamford-New Haven South Mound

151. The Stamford-New Haven South (STNH-S) mound is located 2,200 ft south of the STNH-N mound in the CLIS disposal site. Its location is shown in Figure 13. Once again, because of this mound's proximity to both the FVP and the STNH-N mounds, it has foundation soil profiles and disposal site conditions similar to the FVP and STNH-N mounds.

Background

152. The Stamford-New Haven South and North mounds were constructed as parts of the same dredging project. The STNH-S mound was also included in the New England Division field investigations and monitoring activities undertaken under the DAMOS program. The STNH-S mound was constructed of contaminated

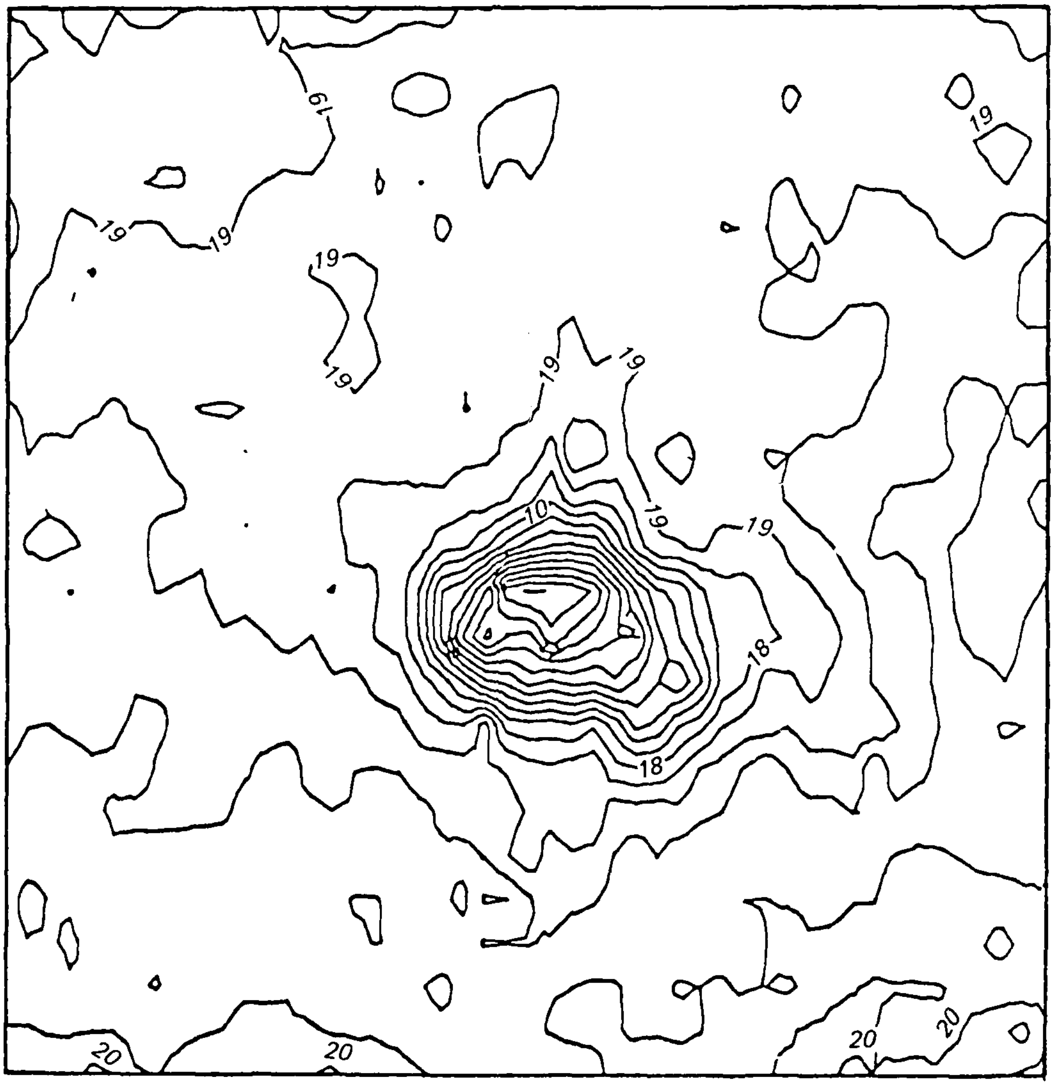

Figure 19. Contour map of STNH-N mound
(contour interval = 1.0 m)

dredged material from the Stamford, CT, harbor. It was subsequently capped with clean silty material from the New Haven, CT, harbor. More details concerning construction of the STNH-S mound are presented in the section on the STNH-N mound (see paragraphs 143-145).

Disposal of material

153. Material from the Stamford, CT, harbor was dredged during March and April 1979 for creation of the STNH-S mound. The material was dredged by clamshell and transported to the disposal site in bottom-dump scows. The scows were individually moved to the disposal site where they discharged their load of dredged material. Disposal at the STNH-S mound location occurred

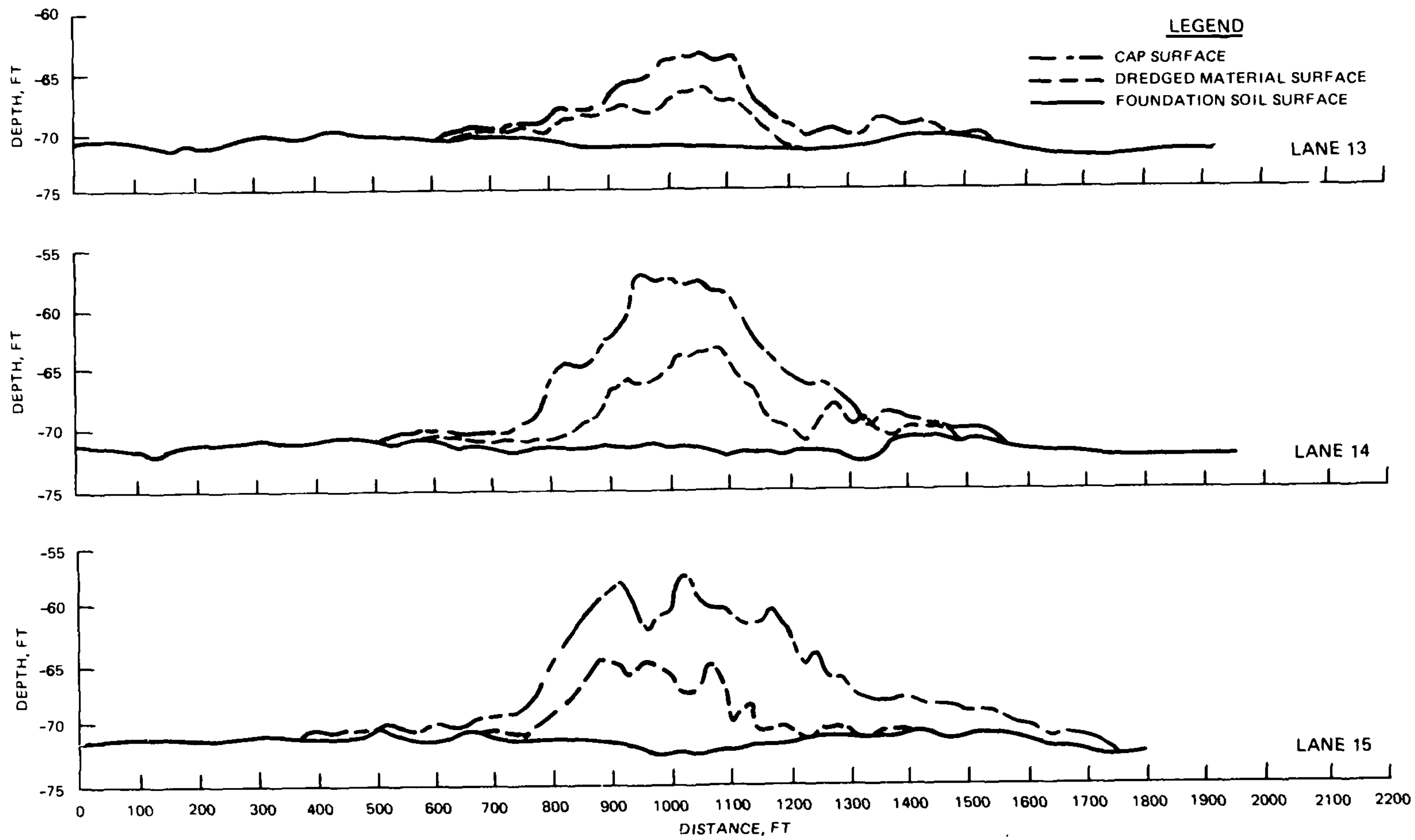

Figure 20. Typical profiles of STNH-N mound

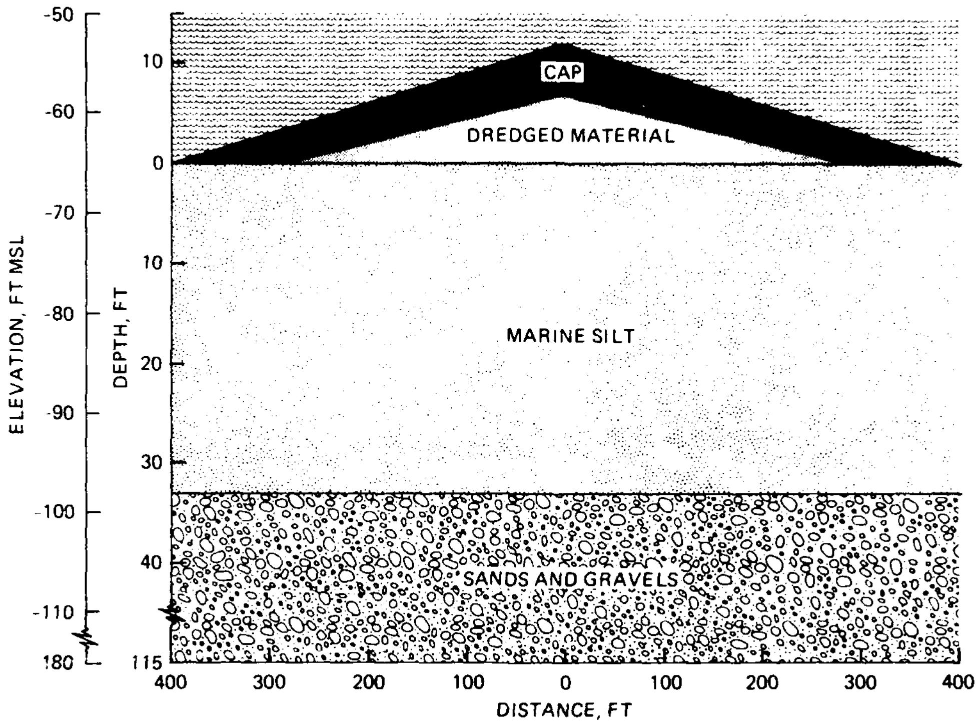

Figure 21. Idealized profile of STNH-N mound and foundation soils

between 25 March and 22 April 1979. The resulting mound contained 49,400 cu yd of contaminated Stamford dredged material and was initially 6.5 ft high.

154. Subsequent to disposal of the contaminated material, capping procedures were initiated. A clamshell dredge was used to remove the silty sediments from the northern end of New Haven harbor. The material was placed in bottom-dump scows and transported to the disposal site. This operation continued from 23 April through 15 June 1979, when all dredging and disposal of fine-grained material was stopped to prevent possible adverse effects on oyster larvae. The capping operation placed approximately 99,300 cu yd of clean silty material on the contaminated Stamford dredged material, resulting in a maximum cap thickness of 12.3 ft.

Sampling and monitoring activities

155. The entire suite of DAMOS sampling techniques was used at the STNH-S mound to monitor its physical, chemical, and biological characteristics, as previously discussed for the STNH-N mound. From the replicate bathemetric

surveys a contour map of the STNH-S mound was developed (Figure 22). Typical profiles of the mound are shown in Figure 23.

156. Diver observations at this mound immediately after placement of the capping material indicated that the silty cap material formed a very uneven surface. Numerous cohesive clumps of material were observed, and the topography of the mound was very irregular. Because of the cohesive nature of this capping material, it did not spread extensively but tended to form a distinct mound that was relatively thick. Therefore, it was more difficult to get complete and uniform coverage of the mound with the silty material.

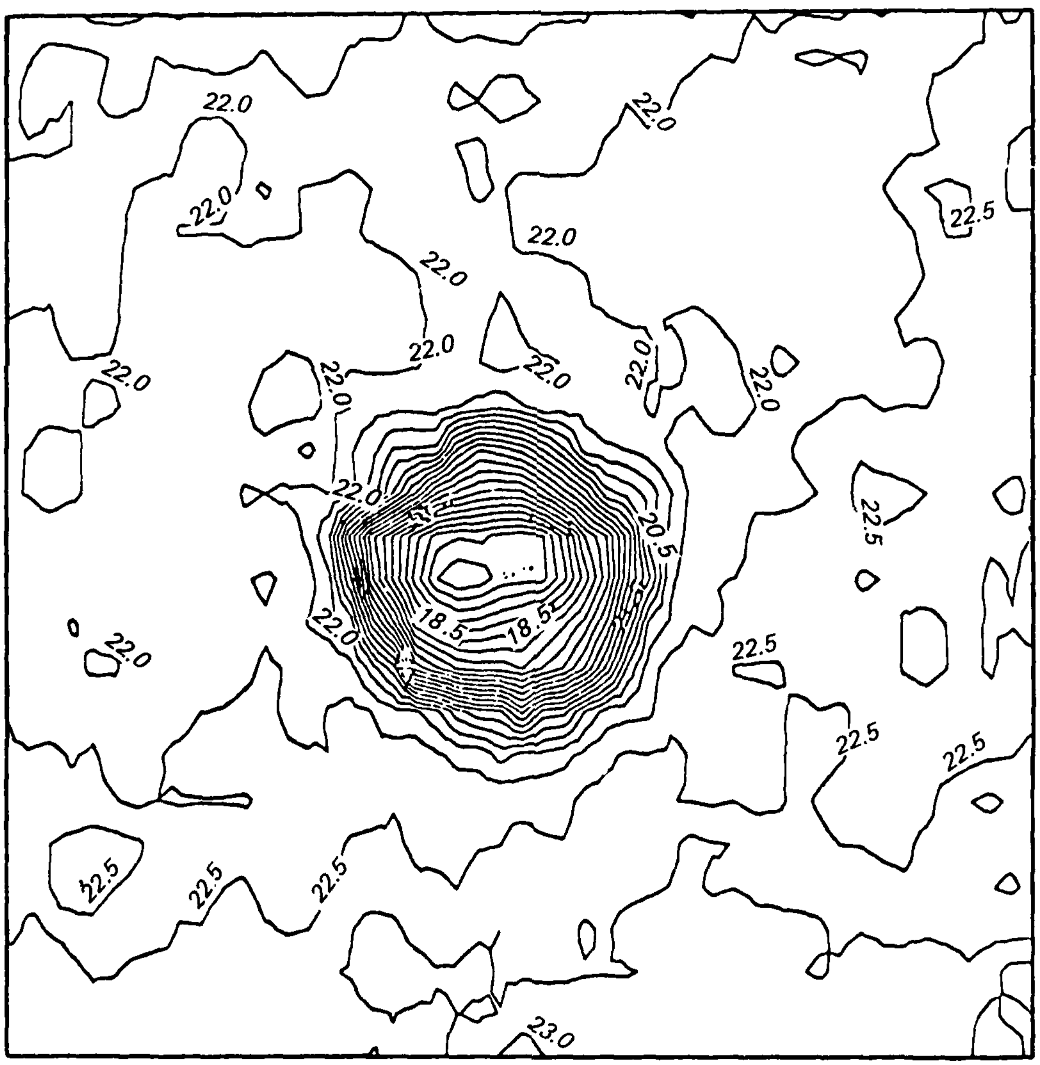

Figure 22. Contour map of STNH-S mound
(contour interval = 0.25 m)

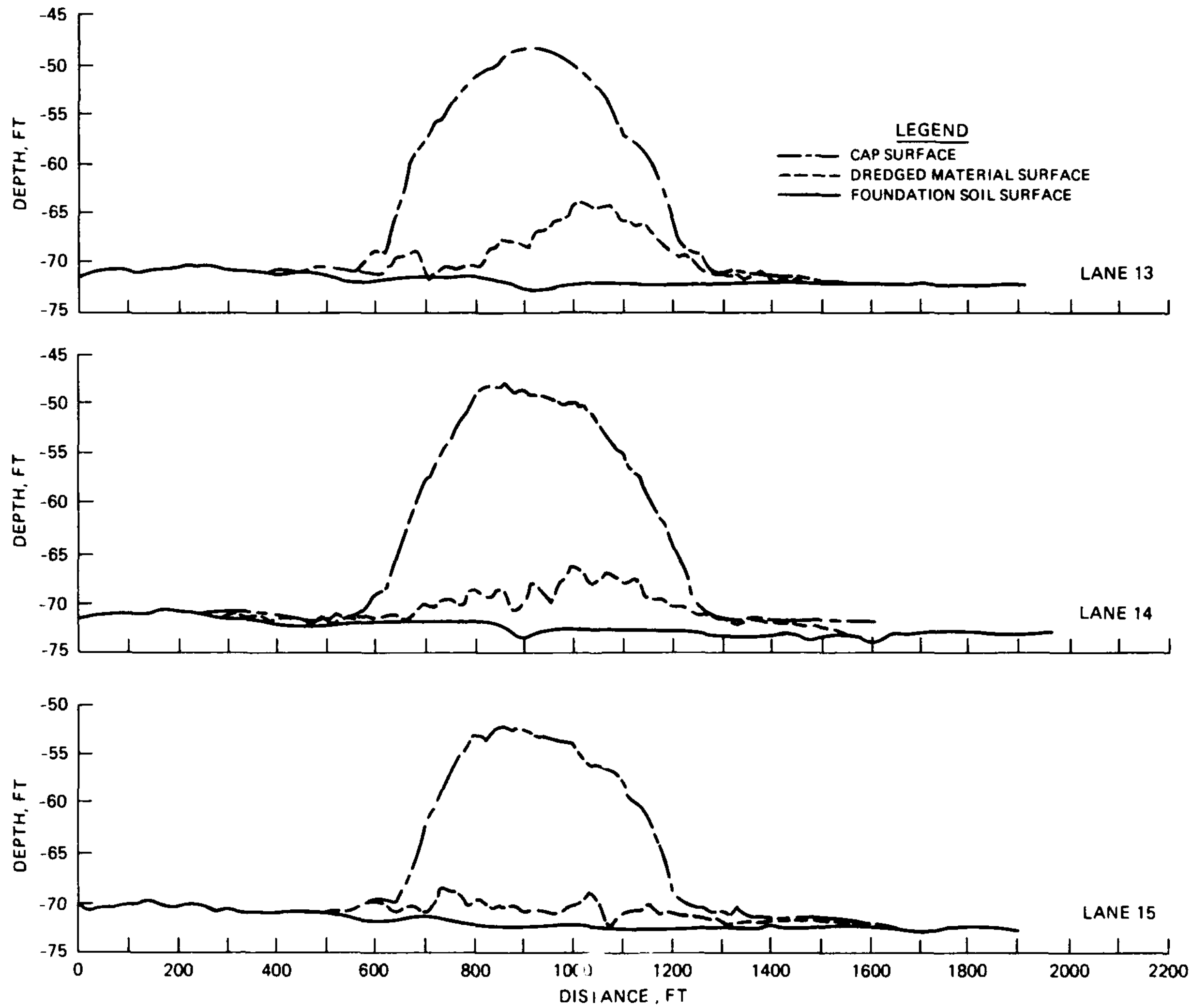

Figure 23. Typical profiles of STNH-S mound

Site conditions

157. The STNH-S mound was constructed of contaminated dredged material from Stamford Harbor that was capped with clean silt. The resulting mound had a height of contaminated material of 6.5 ft and a cap thickness of 12.3 ft. Thus, the total mound height at completion of disposal was 18.8 ft; the mound had a diameter of 660 ft. This material was placed on a foundation consisting of 33 ft of marine silt which was overlying 82 ft of sands and gravels. Figure 24 presents the mound and foundation soil profile that will be used in subsequent analyses.

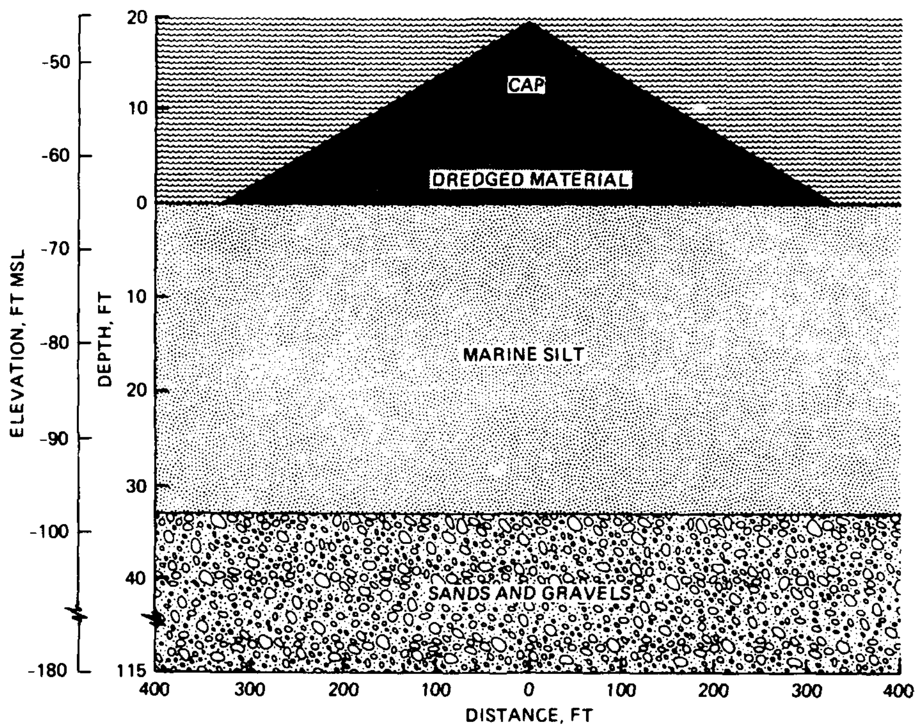

Figure 24. Idealized profile of STNH-S mound and foundation soils

PART VI: LABORATORY TESTING

158. Use of the computer program MOUND to predict the consolidation of dredged material mounds by finite strain theory requires input data that must be determined through a geotechnical laboratory testing program. The necessary data include specific gravity of the solid particles, the void ratio-effective stress ($e - \sigma'$) relationship, and the void ratio-permeability ($e - k$) relationship. The specific gravity can be determined by routine laboratory testing, while the $e - \sigma'$ and $e - k$ relationships must be determined from one or more of a number of laboratory consolidation tests.

159. For determining the $e - \sigma'$ and $e - k$ relationships, several consolidation test procedures are available. The laboratory consolidation tests used by the Corps of Engineers for dredged material testing include the standard oedometer test, the self-weight consolidation test, and the large strain, controlled rate of strain (LSCRS) test. Additionally, a new consolidation test device and procedure for use in dredged material testing are currently under development; the test is referred to as the controlled rate of strain (CRS) test. The actual testing procedures, with their associated advantages and disadvantages, are discussed in detail elsewhere (Poindexter 1987, 1988). For the stiffer foundation soils, standard oedometer tests will typically provide adequate data. In the following paragraphs, results of the laboratory testing program are presented.

160. Both the void ratio-effective stress and the void ratio-permeability relationships, which are required for calculation of consolidation by the finite strain theory, must be developed from the laboratory tests for each material. These relationships should extend across the entire range of void ratios that may exist in the dredged material. Therefore, results obtained from the various consolidation tests are combined to yield composite $e - \sigma'$ and $e - k$ relationships. For input to the computer program MOUND, these data are described by equations of the form $e = A\sigma^{B} + C$ and $e = Dk^{E} + F$, where the coefficients are determined from laboratory data for each individual sediment. Several curve fit models were evaluated for use, and the above relationships consistently fit the data better.

161. Additional soil properties needed for complete analysis by the computer program MOUND include specific gravity of solids, Atterberg limits, and

the activity, A , of material. All of these items are determined or can be derived from standard laboratory engineering classification tests.

162. In the following paragraphs, the material properties for each of the dredged materials of interest in this study are presented. Because the new CRS consolidation test is still under development, data from this test are simply plotted for comparison purposes and are not used for material characterization in this study. The curve plotted on each $e - \sigma'$ and each $e - k$ figure is the curve fitted to those data by the equations discussed in paragraph 160.

Duwamish Waterway Materials

163. Classification tests were run on the contaminated dredged material using the composite sample collected from the transport barge during dredging. This material had a liquid limit of 73 and a plastic limit of 34, yielding a plasticity index of 39; the natural water content of the material was 90.9. This material had a specific gravity of 2.48. The material was classified according to the Unified Soil Classification System (USCS) as a black sandy clay of high plasticity (CH). Results of the classification tests are given in Appendix A.

164. Using the gradation curve for the Duwamish Waterway sediment, the activity of the material was calculated. The activity, which is equivalent to the plasticity index divided by the percent clay in the sample (i.e., the material finer than 0.002 mm), was found to be 3.71. Consolidation tests run on the Duwamish Waterway contaminated dredged material included the standard oedometer, self-weight, and LSCRS tests; the developmental CRS test was also performed on this material. Results of the oedometer, self-weight, and LSCRS tests were combined to form the total $e - \sigma'$ and $e - k$ relationships for this material, as shown in Figures 25 and 26, respectively. Problems associated with data from the new test precluded its inclusion in this analysis; these problems are discussed at length by Poindexter (1987, 1988).

165. Classification tests were also run on a composite sample of the capping material; this composite sample was also collected during dredging and filling of a transport barge. The capping material was found to be a dark gray silty sand (SM) with a specific gravity of 2.78 and a natural water

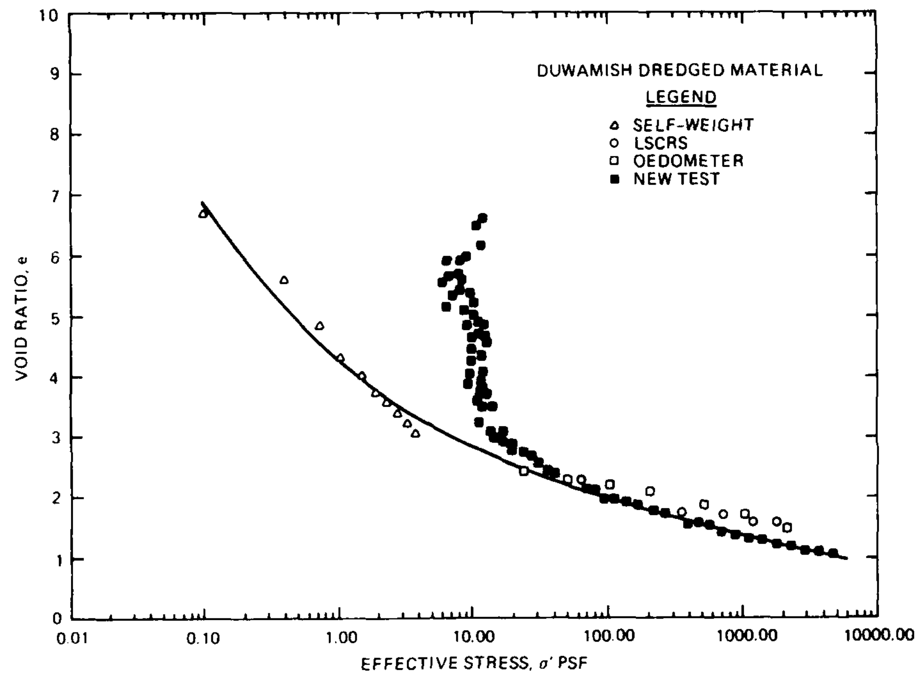

Figure 25. Void ratio-effective stress relationship for Duwamish Waterway dredged material

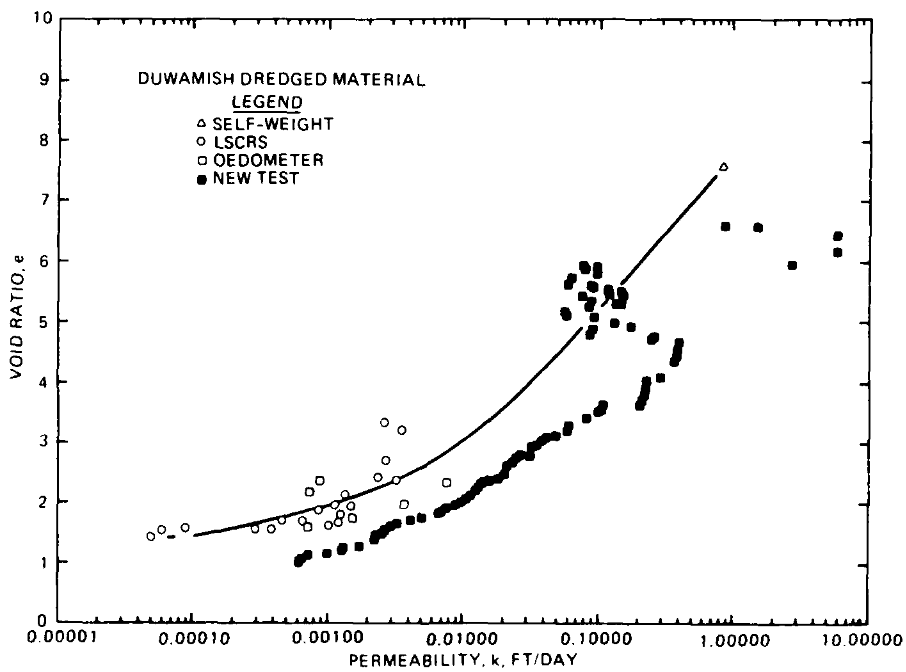

Figure 26. Void ratio-permeability relationship for Duwamish Waterway dredged material

content of 61.7. Results of classification tests on the Duwamish Waterway capping material are shown in Appendix A.

Field Verification Program Materials

166. Samples of contaminated sediment to be placed to form the Field Verification Program mound were collected from Black Rock Harbor prior to dredging. Classification tests run on a composite material indicated that it had a plasticity index of 66, with a liquid limit of 112 and a plastic limit of 46. The specific gravity of this material was 2.56, and the natural water content was 252.6. According to the USCS, the FVP sediment was classified as a black sandy clay (OH). The results of classification testing are shown in Appendix A. The activity of the dredged material was calculated to be 4.4.

167. The oedometer test, self-weight consolidation test, and the new CRS test were run on the FVP dredged material to obtain the necessary compressibility and permeability data. Additionally, a constant rate of deformation test (CRDT) was performed by the University of Colorado; permeability values were calculated for this test at the drained boundary, db , and from the finite strain coefficient of consolidation, g . The $e - \sigma'$ and $e - k$ relationships obtained from these tests are shown in Figures 27 and 28, respectively.

168. Because the foundation soils at the Central Long Island Sound disposal site include some compressible marine sediments, laboratory tests were conducted on this material. Classification tests conducted on the compressible foundation soil indicated that it had a natural water content of 95, liquid limit of 80, plastic limit of 35, and plasticity index of 45. The specific gravity was 2.72. The activity of the foundation soil was calculated to be 1.80. Results of the classification tests are given in Appendix A. A standard oedometer test was conducted on this material, and results are presented in Figures 29 and 30.

Stamford-New Haven North Materials

169. Samples of the contaminated dredged material were obtained from the Stamford-New Haven North (STNH-N) mound several years after construction of this mound. The various core samples were combined to form a composite

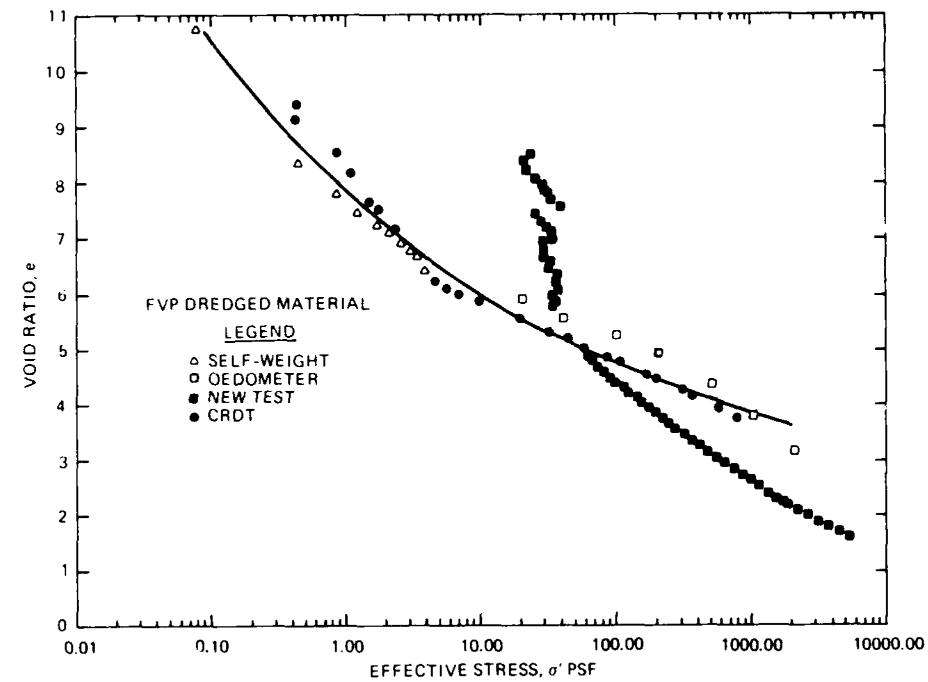

Figure 27. Void ratio-effective stress relationship for FVP dredged material

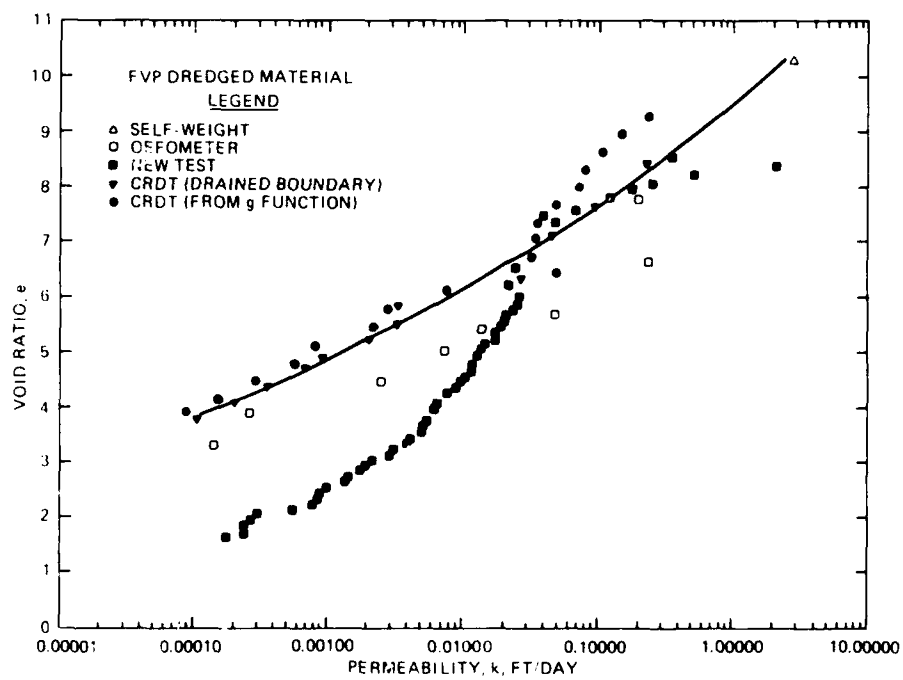

Figure 28. Void ratio-permeability relationship for FVP dredged material

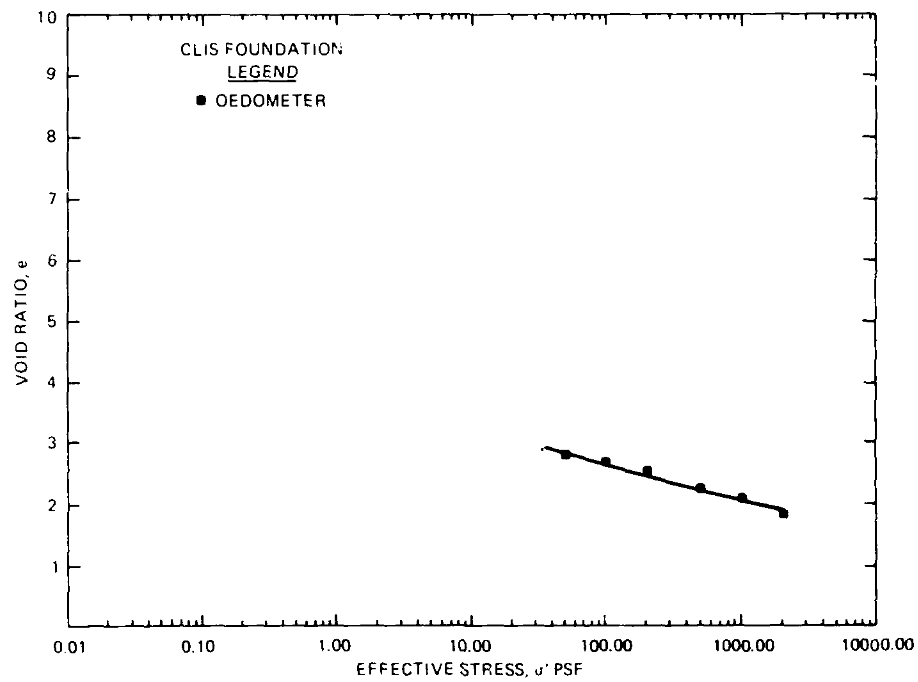

Figure 29. Void ratio-effective stress relationship for Central Long Island Sound foundation soil

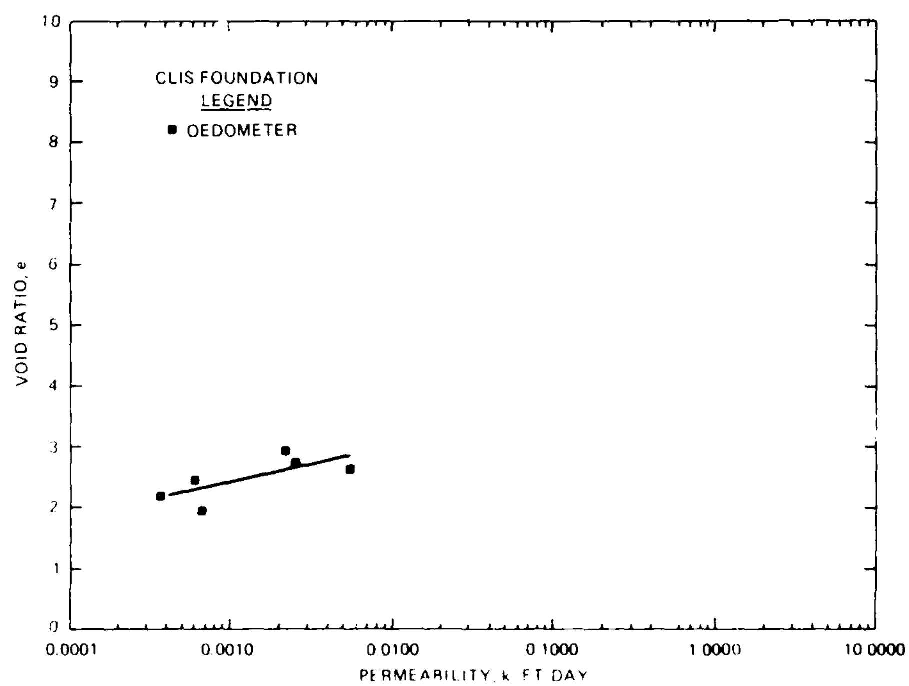

Figure 30. Void ratio-permeability relationship for Central Long Island Sound foundation soil

material for laboratory testing. Engineering classification tests on this material indicated that the dredged material had a liquid limit of 85 and a plastic limit of 40; thus, the plasticity index of the material was 45. The specific gravity was 2.72. The activity of this material was calculated to be 1.25. Classification test results for the Stamford dredged material are presented in Appendix A.

170. Consolidation tests performed on the STNH-N dredged material included the oedometer test, the self-weight consolidation test, and the new CRS test. The compressibility and permeability relationships developed from the laboratory tests are shown in Figures 31 and 32, respectively.

171. Classification tests were performed on the STNH-N capping material. This material consisted of 98 percent sand and 2 percent silt; it was therefore nonplastic. Results of classification tests for the New Haven sand capping material are shown in Appendix A.

172. The foundation soil at the STNH-N mound location is the same as that for the FVP mound. Classification data and relationships for $e - \sigma'$ and $e - k$ are presented in, respectively, Appendix A and Figures 29 and 30.

Stamford-New Haven South Materials

173. The material dredged from Stamford Harbor was deposited to form both the Stamford-New Haven South and North mounds. Therefore, classification and consolidation test results presented in Appendix A and Figures 31 and 32 are also to be used for the STNH-S dredged material.

174. The silty capping material at the STNH-S mound was subjected to classification and consolidation testing. Classification tests indicated that the capping material had a plasticity index of 45 (liquid limit 86, plastic limit 41). The natural water content was 85, and the activity was calculated to be 1.96. Results of the classification testing are presented in Appendix A.

175. Both the LSCRS and self-weight consolidation tests were run on the STNH-S capping material. The resultant compressibility and permeability relationships are shown in Figures 33 and 34, respectively.

176. The foundation soil at the STNH-S mound was identical to that at the other CLIS disposal sites. Thus, Appendix A and Figures 29 and 30 present, respectively, the classification, $e - \sigma'$, and $e - k$ data for the STNH-S foundation soil.

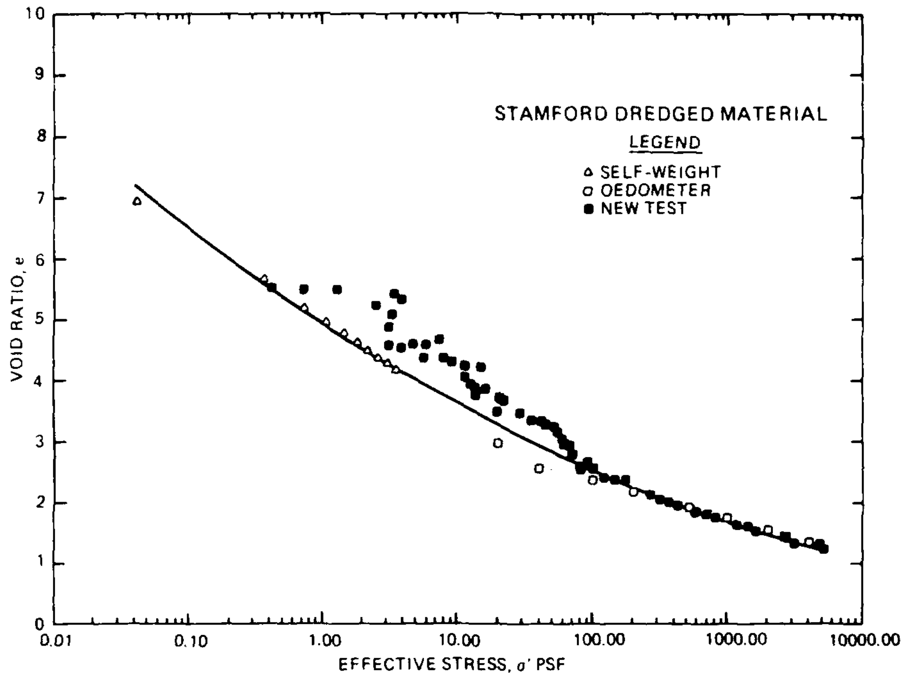

Figure 31. Void ratio-effective stress relationship for Stamford dredged material

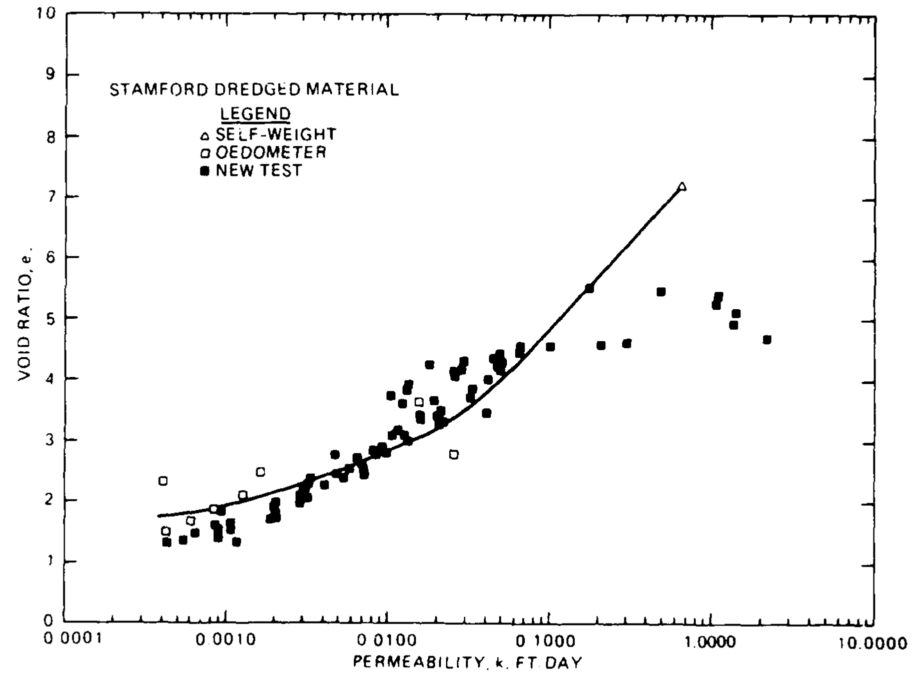

Figure 32. Void ratio-permeability relationship for Stamford dredged material

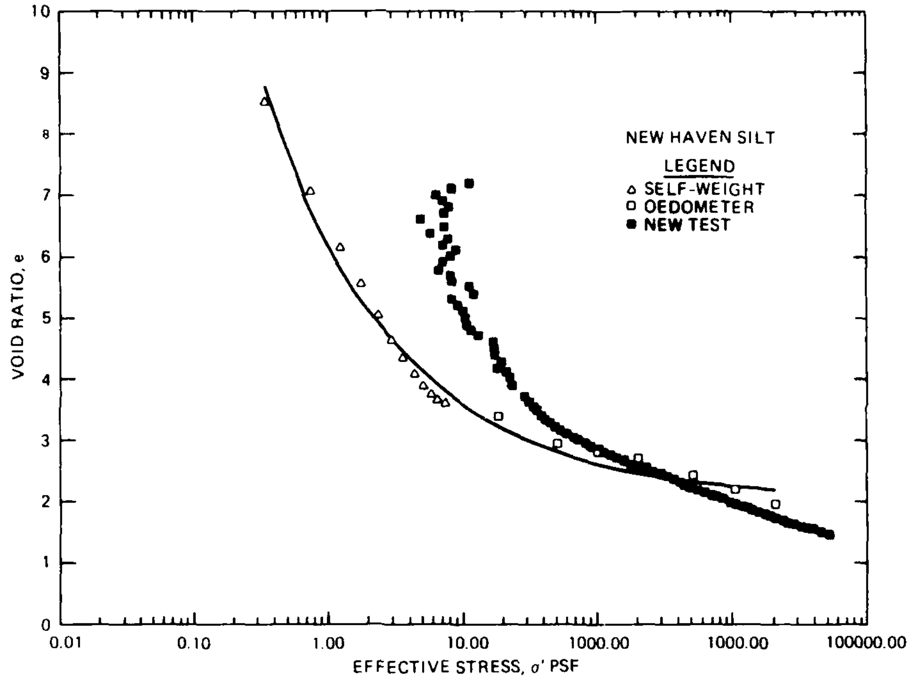

Figure 33. Void ratio-effective stress relationship for New Haven silt capping material

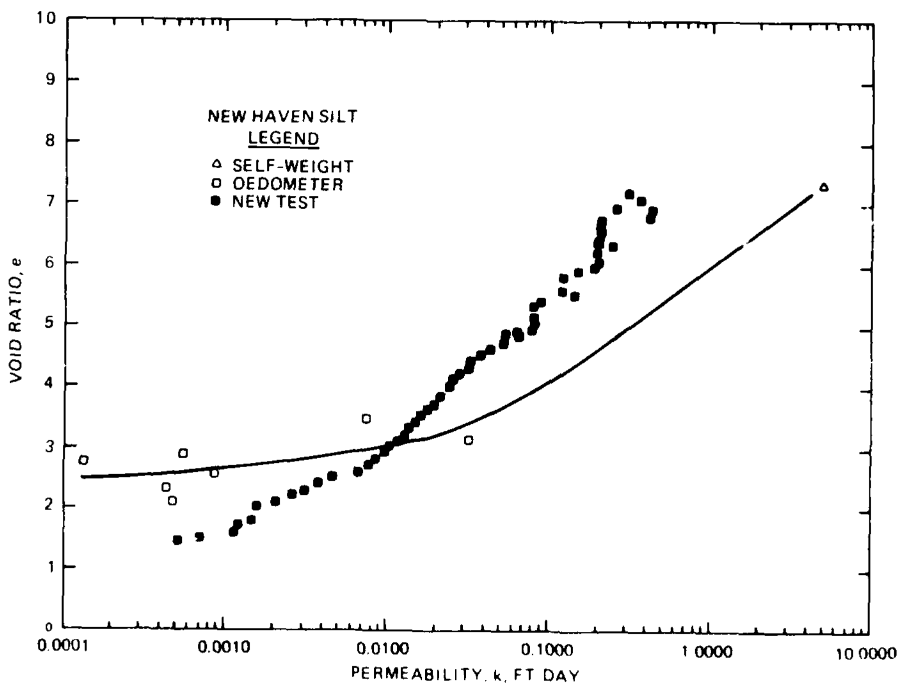

Figure 34. Void ratio-permeability relationship for New Haven silt capping material

PART VII: COMPARISON OF MEASURED WITH PREDICTED PERFORMANCE

177. The dredged material mounds, described previously, were analyzed for consolidation behavior using the computer program MOUND. The analysis provided information on void ratio, effective stress, and pore water pressure changes with time, as well as information on the rates of consolidation. Estimates of shear strength as consolidation proceeds were also obtained.

178. Results of the predicted settlements were then compared with measured field behavior to determine the accuracy of the predictive technique. The types of field monitoring, as well as the frequency of monitoring, varied considerably from site to site. Therefore, the level of detail of field behavior available to compare with the predicted behavior also varied significantly.

179. In the following paragraphs, results of the performance predictions made for each dredged material mound by using the computer program MOUND are discussed. The predicted performance is then compared with the actual performance for each field site.

Duwamish Waterway Mound

180. The Duwamish Waterway mound had the initial cross-sectional configuration of a truncated cone 6 ft in height, 305 ft in diameter, with side slopes of approximately 1V:17H. The mound was composed of 3 ft of compressible dredged material with side slopes of 1V:20H, overlain by a sand cap.

181. The material properties reported in Part VI for the Duwamish Waterway were used to determine the predicted consolidation behavior of the mound. Since both the foundation soil and the capping material were composed of sand, consolidation occurred only in the contaminated dredged material. The unit weight of the sand cap was calculated using the field-determined water content of the cap; the saturated unit weight was calculated to be 103 pcf. Because of potential problems with water content and density determinations from Vibracore samples and because the density value seemed low, comparison was made between this value and typical values for a mixed-grained sand (Terzaghi and Peck 1967). The density of a sand such as the Duwamish capping sand was expected to be 124 pcf. For this analysis, the average of the field-determined density and the typical density was used, although all three density values were used to obtain comparative settlement versus time plots.

182. Consolidation of the dredged material mound was simulated with the computer program MOUND. The effect of the sand cap was modeled by using a surcharge load calculated from a cap density of 113.5 pcf. One hundred-percent primary consolidation was predicted to occur within 180 days of mound creation. This relatively rapid completion of settlement and dissipation of all excess pore water pressures is attributed to the thinness and density of the compressible layer in conjunction with the presence of a significant surcharge load and location of the compressible material between two sand layers.

183. The progression of consolidation in the compressible dredged material is illustrated in Figure 35 where the profiles of void ratio, effective stress, and excess pore pressure are shown for various times. The predicted development of shear strength over time is also shown in Figure 35. The effect of double drainage can be seen in this figure as the excess pore pressures dissipate, effective stresses increase, and the void ratios decrease at both the top and bottom of the consolidating layer. In this mound, the surcharge load does not appear to have caused formation of a less permeable filter-cake layer at the top of the consolidating material, as has been observed for other sites; this is not unexpected behavior since the material was placed at a relatively low void ratio and underwent only a small amount of consolidation, resulting in a small amount of drainage and accompanying potential particle migration.

184. The predicted settlement with time is compared with the measured settlement plate data in Figure 36. For this comparison, the actual thickness (30 in.) of material layers at the settlement plate, instead of the assumed mound geometry (36 in.), was used in the predictions; settlement predictions were made for cap densities of 103, 113.5, and 124 pcf, as discussed earlier. The field observations compare well with the range of predicted settlements for the Duwamish mound. As shown in Figure 36, the investigated variation in density of the sand cap caused an ultimate settlement of 0.20 to 0.26 ft, with ultimate measured field settlement being approximately 0.25 to 0.27 ft. The maximum difference in predicted ultimate settlement of 0.06 ft is within the expected range of experimental error for diver readings of settlement plates. Thus, it is difficult to determine which prediction best fits the observed field data, but it can be concluded that the prediction matches the field data well.

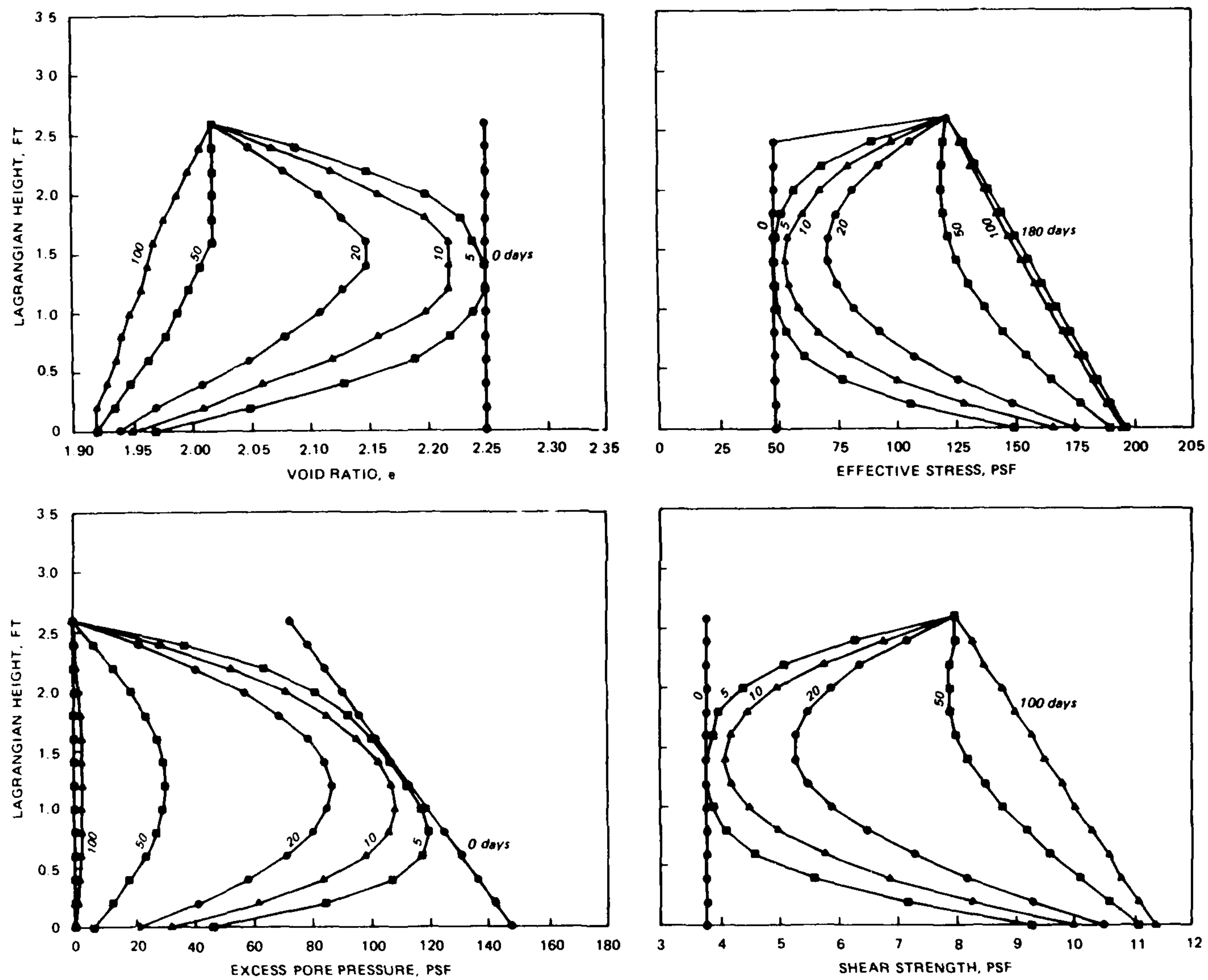

Figure 35. Temporal profiles of void ratio, effective stress, excess pore pressure, and shear strength for Duwamish Waterway mound

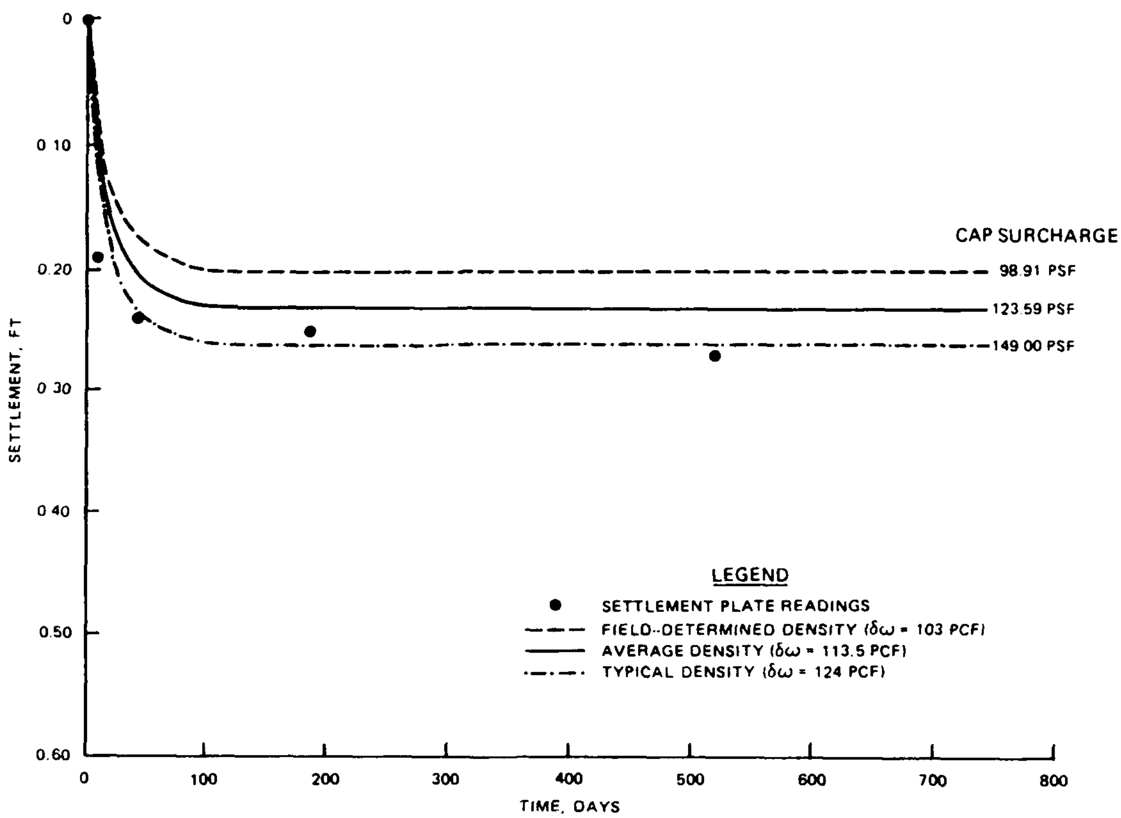

Figure 36. Time-rate of consolidation at settlement plate location for Duwamish Waterway mound

185. The predicted final configuration of the Duwamish Waterway mound is shown in Figure 37. Because this mound underwent relatively little consolidation and there was no foundation consolidation, the initial and final mound configurations are similar. In this particular case, consideration of the postformation consolidation behavior of the mound does not significantly affect or alter the remaining physical capacity of the disposal site. This would not necessarily be true for thicker, less dense mound deposits.

Field Verification Program Mound

186. The FVP mound was initially conical in shape, being 6.6 ft high and 660 ft in diameter. The side slopes of this mound were approximately 1V:50H. No capping material was placed on this mound.

187. The consolidation properties for the FVP dredged material and foundation soils were reported in Part VI. These compressibility and permeability

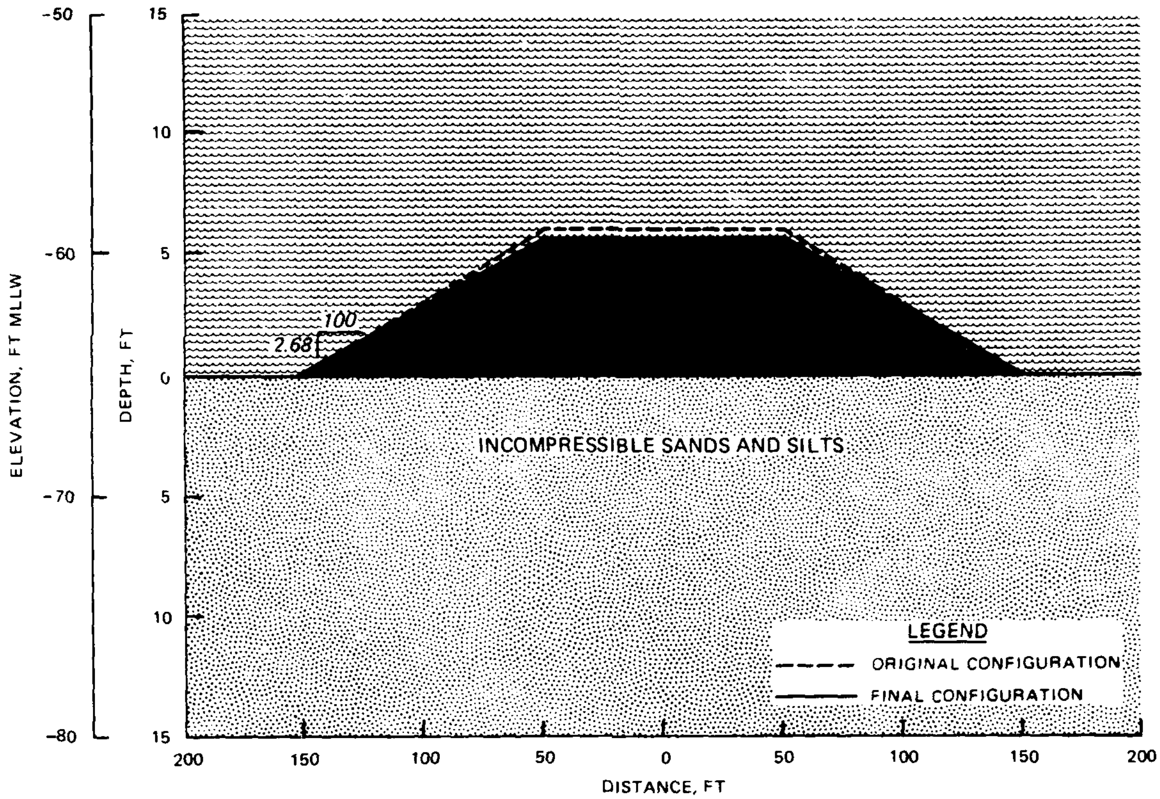

Figure 37. Predicted mound configuration at completion of consolidation of Duwamish Waterway mound

relationships were used in the computer program MOUND to predict the consolidation behavior of the FVP mound.

188. Predictions indicated that 100-percent primary consolidation occurred approximately 5,040 days after formation of the mound, although 90-percent consolidation occurred at about 800 days and 95-percent occurred at 1,260 days. The long time required to reach 100-percent consolidation resulted because consolidation is driven only by the self-weight of the soft dredged material; if a surcharge were placed on this mound, 100-percent consolidation would occur much more quickly.

189. The progression of consolidation in the FVP mound is shown in Figure 38 by advancement of the void ratio, effective stress, excess pore water, and shear strength profiles through time. As shown in this figure, changes in the various profiles occur very slowly, indicating the slow rate of consolidation of this mound. It should be noted that only an insignificant change in shear strength is indicated for the FVP mound. This is consistent with

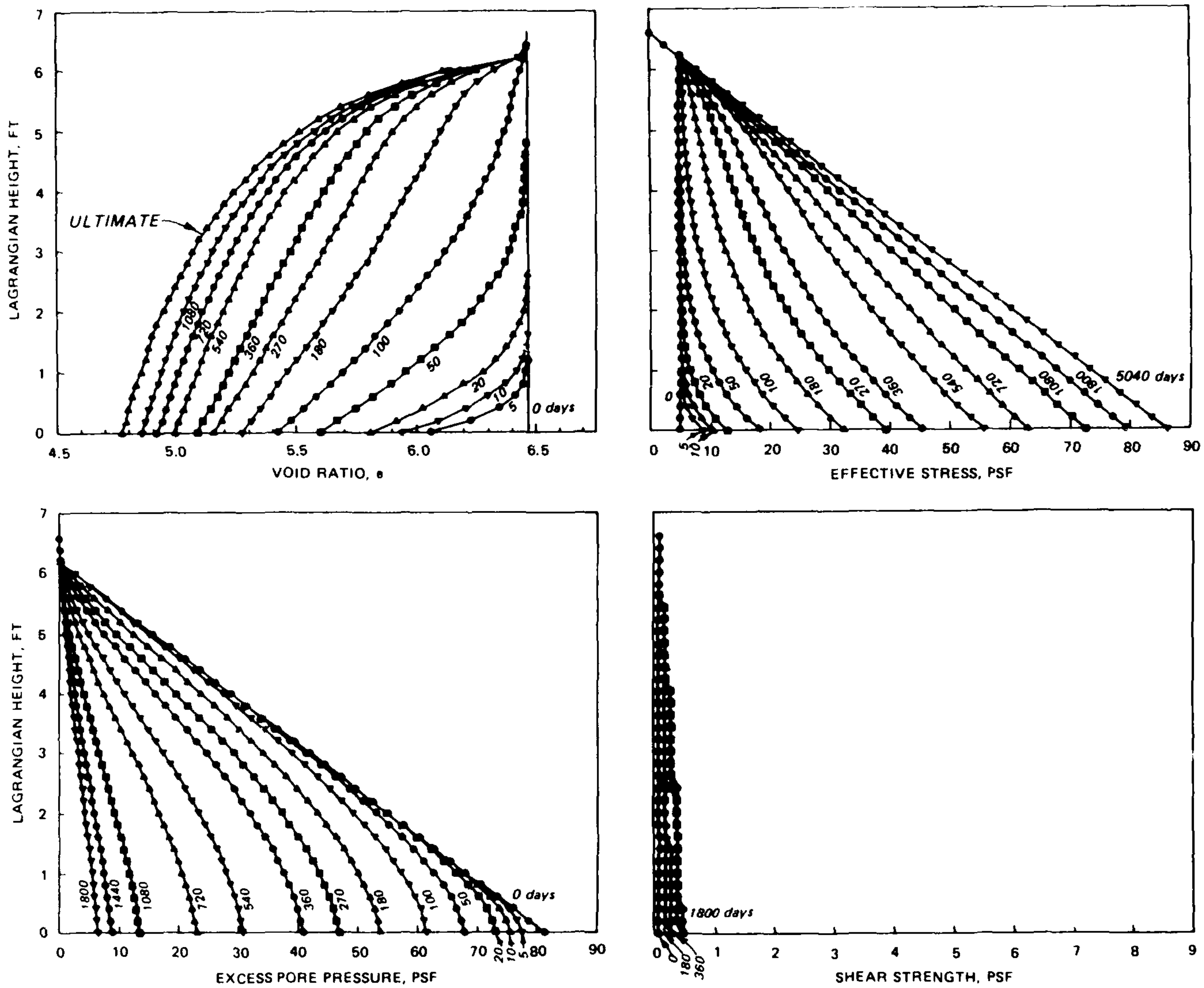

Figure 38. Temporal profiles of void ratio, effective stress, excess pore pressure, and shear strength for FVP mound

previous observations of dredged material placed in confined upland disposal sites where the only noticeable change in shear strength typically occurs within the desiccation crust when ponded surface water is removed and evaporative drying occurs. Because the only loading to which the FVP material is subjected is that of self-weight and since the material is in a subaqueous environment where no evaporative drying can occur, the shear strength would be expected to be relatively low and uniform throughout the depth of the deposit. Although further field studies are needed to verify the accuracy of the predicted values, the relative values of shear strength are believed to be indicative of relative field values at the FVP mound.

190. For the FVP mound, the predicted settlement with time is compared with field observations in Figure 39. The shaded vertical bands in this figure indicate the precision of the hydrographic survey data; the solid circles represent the mean value. Very close agreement was obtained between the predicted total settlement and a number of the actual field settlement observations calculated from hydrographic survey data. Considering the irregular surficial geometry of subaqueous mound deposits in conjunction with the vertical and horizontal imprecision typically associated with oceanic hydrographic surveying, the agreement obtained between field observation and analytical prediction is considered excellent.

191. The final configuration of the FVP mound, as determined from analytical predictions, is shown in Figure 40. The center height of the mound is expected to decrease from 6.6 ft to 4.85 ft above the original foundation elevation. Combined settlement of the foundation soil and the mound results in a gain in remaining storage capacity of 7,400 cu yd above that existing immediately after mound formation. For a mound that was initially only 6.6 ft thick, this represents a 26.5-percent reduction in required storage volume for the existing mound and translates directly to a gain in remaining storage capacity. Thus, the potential significance of considering the physical reduction in size of a subaqueous disposal mound resulting from consolidation becomes apparent.

Stamford-New Haven North Mound

192. The STNH-N mound had an initial maximum height of 11.80 ft with a diameter of 810 ft; the side slopes of this conical mound were 1H:34V. The

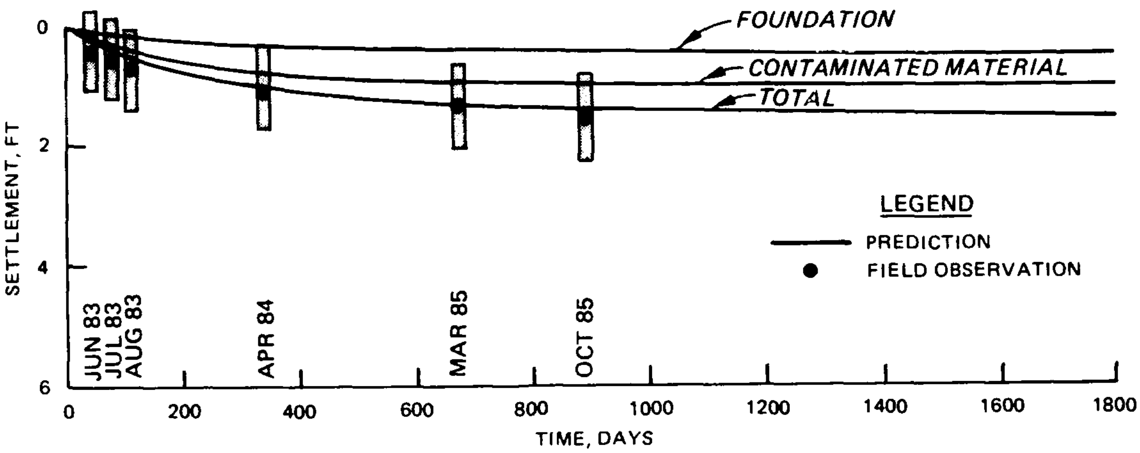

Figure 39. Time-rate of consolidation at center of FVP mound

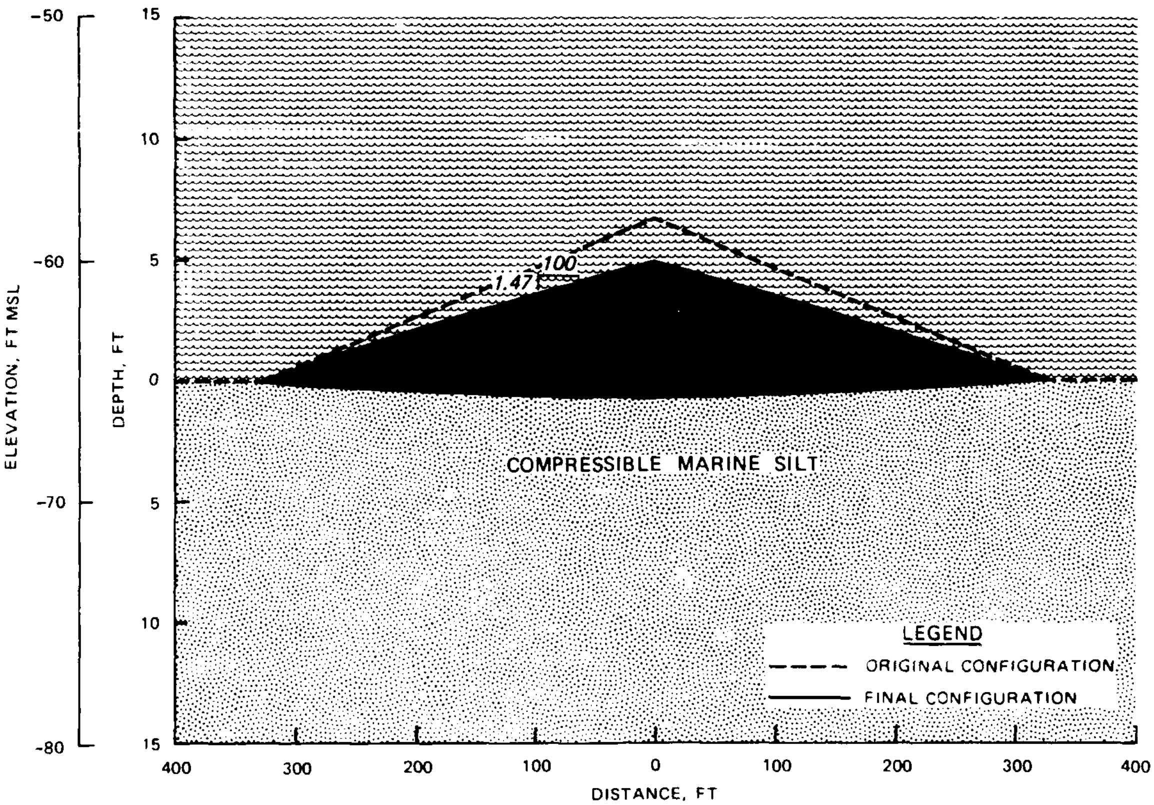

Figure 40. Predicted mound configuration at completion of consolidation of FVP mound

core of the mound was composed of contaminated material that was 7.0 ft thick at the center and had side slopes of 1H:41V; the core was covered with clean sand.

193. Material properties for the STNH-N mound were determined and reported in Part VI. These properties were used in the computer program MOUND to predict the behavior of the STNH-N mound.

194. For the STNH-N mound, completion of consolidation within the mound itself was predicted to occur within 3 years after mound creation; 90-percent consolidation of the dredged material was predicted to occur within 100 days, and 98 percent was predicted within 6 months. Foundation consolidation would require more than 10 years to reach 100-percent primary consolidation. When primary consolidation was completed within the mound, excess pore pressures of 14.8 psf were still present at the base of the mound; these pore pressures were maintained for a significant time by drainage from the foundation soil. Excess pore pressures existing after completion of consolidation are typically predicted by finite strain consolidation theory because of the particular definition of degree of consolidation used in this theory, as discussed in Part IV.

195. Decreases in void ratio, accompanied by dissipation of excess pore pressures, subsequent increases in effective stress, and predicted increases in shear strength within the dredged material mound are illustrated in Figure 41. Large changes in these profiles occur quickly during the early stages of consolidation because of the large surcharge caused by the sandy capping material. It should be noted that a filter-cake of less permeable material develops as the uppermost portion of the dredged material consolidates rapidly during the early stages. The presence of the filter-cake is demonstrated by a very dramatic decrease in excess pore pressure of approximately 300 psf across a Lagrangian thickness of 0.5 ft, which corresponds to an actual (Eulerian) height of approximately 0.2 ft. The same phenomenon has been observed in the LSCRS test when a dimensionally thick slurry sample was subjected to rapid loading. In the laboratory, a filter-cake of dense material with low permeability formed at each drainage boundary. Therefore, during the early stages of consolidation in both the laboratory and in the field, slurried material subjected to rapid, heavy loading exists as a heterogeneous mass that becomes more homogeneous as consolidation progresses into the interior of the material.

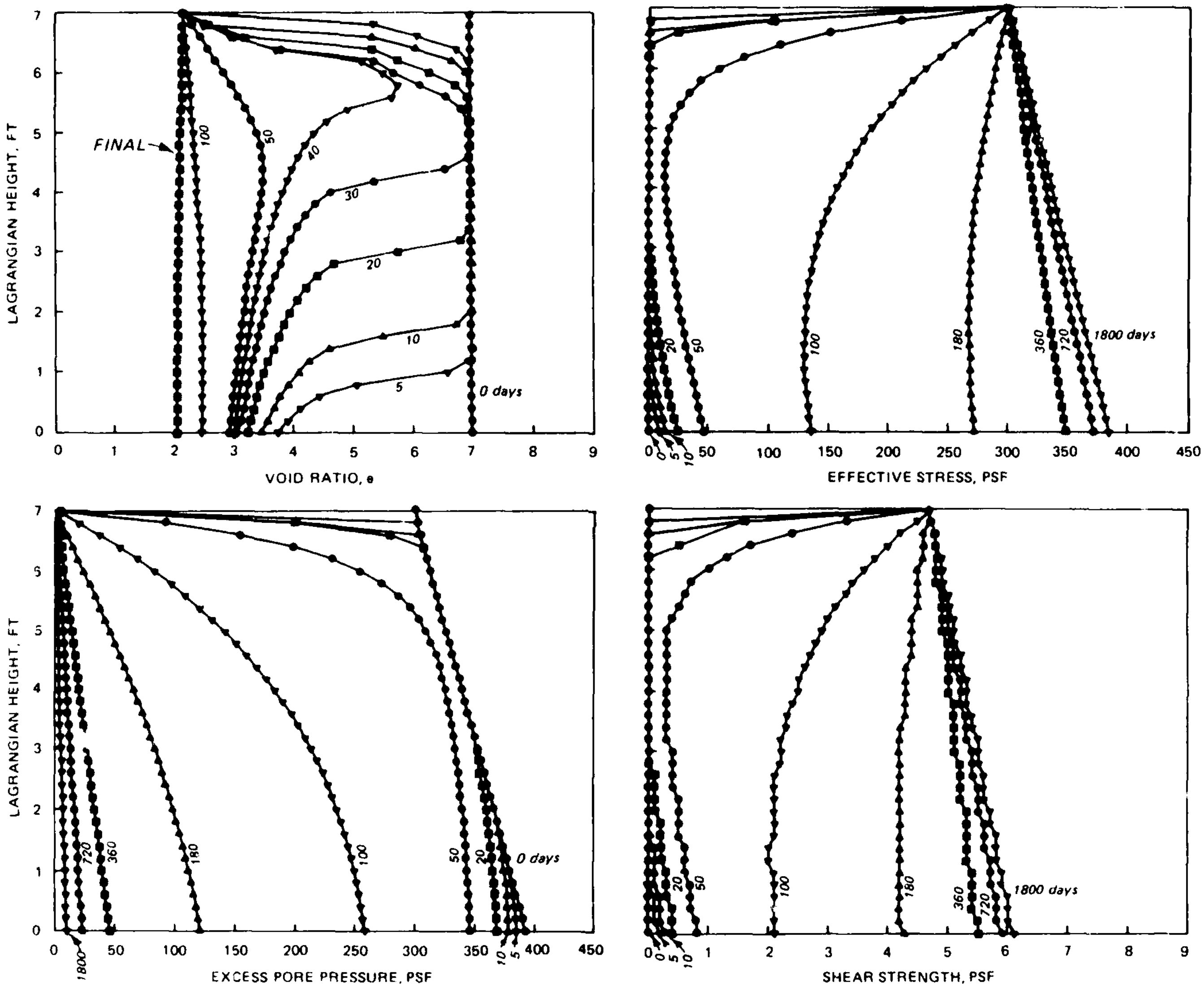

Figure 41. Temporal profiles of void ratio, effective stress, excess pore pressure, and shear strength for STNH-N mound

196. In Figure 42, the predicted settlement with time is compared with actual field data from the STNH-N mound. It should be noted that the starting point for the settlement prediction was taken from hydrographic survey data, thus subjecting it to the same ±0.7-ft survey precision as the field data points. When the precision of the prediction and the precision of the data points are considered together, it is felt that very good overall agreement exists between the predicted and observed settlement values.

197. The predicted final configuration of the STNH-N mound is shown in Figure 43. The maximum height of the mound will decrease from 11.9 ft above the original foundation elevation to 5.86 ft as a result of both dredged material and foundation soil consolidation. This will cause an increase in remaining storage capacity of the disposal site of 38,400 cu yd due completely to consolidation. This represents approximately a 51-percent reduction in disposal site storage volume needed for the existing mound. This demonstrates that accounting for postdisposal consolidation of a dredged material mound can significantly increase the projected remaining disposal site capacity.

198. When disposing of contaminated dredged material, a cap of clean material must be used to isolate the contaminants from the overlying environment, prevent intrusion of burrowing organisms into the contaminated material, and

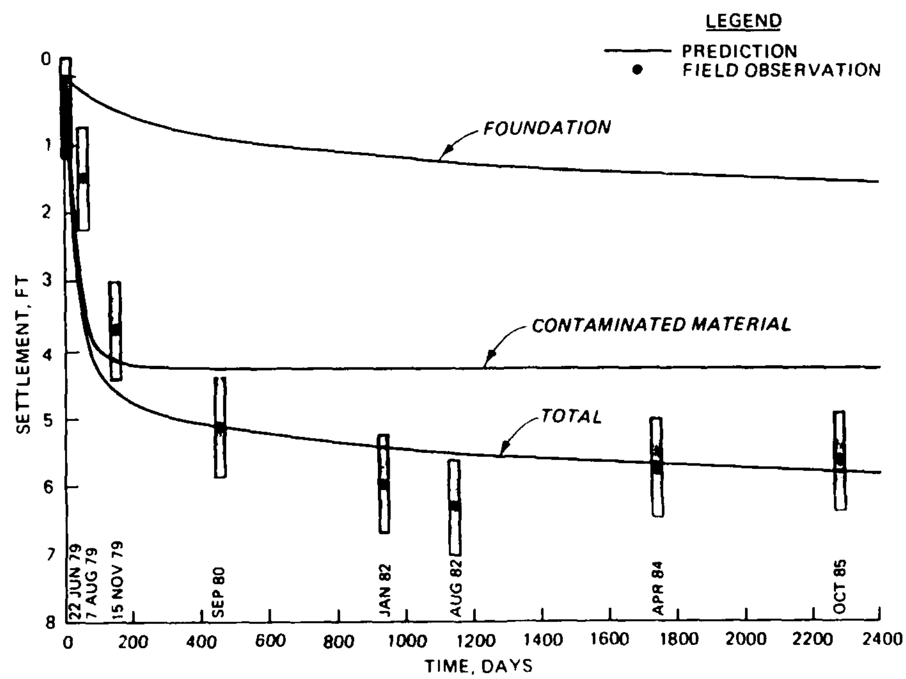

Figure 42. Time-rate of consolidation at center of STNH-N mound

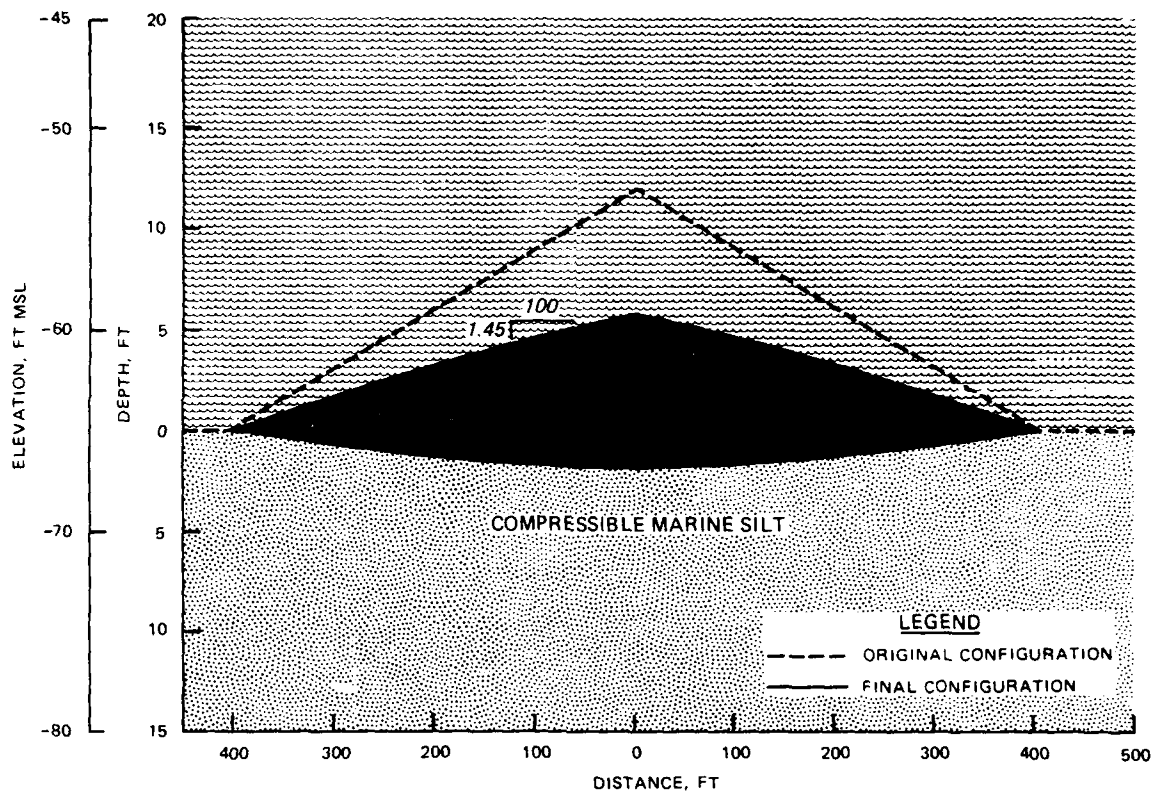

Figure 43. Predicted mound configuration at completion of consolidation of STNH-N mound

protect the contaminated material from erosion, resuspension, and transport from the disposal site. However, placement of such a cap typically consumes a significant amount of the disposal site. In the case of the STNH-N mound, the cap occupied 2.43 times the volume originally required for the contaminated dredged material. This at first seems to represent, from a volumetric standpoint, inefficient and unwise use of a disposal site with a fixed capacity, but examination indicates that the cap causes considerable consolidation of both the dredged material and the underlying foundation soil. For the STNH-N mound, the final disposal site volume that is occupied by the entire mound (dredged material plus cap) is 1.69 times the initial volume of the contained dredged material alone, indicating that use of a cap is not as detrimental to disposal site capacity as it might seem. Thus, the large reduction in occupied disposal site volume resulting from consolidation also significantly reduces the negative volumetric impact of cap placement on disposal site capacity.

Stamford-New Haven South Mound

199. The conical STNH-N mound was initially composed of a core of contaminated dredged material 6.6 ft high and 600 ft in diameter, with side slopes of 1V:45H. This material was covered with a silt capping material to create a mound with overall dimensions of 18.8-ft height, 600-ft diameter, and 1V:18H side slopes.

200. Properties for materials placed in the STNH-S mound were reported in Part VI. These material properties were used in the computer program MOUND to predict the field behavior of the mound. Because MOUND, as well as other presently available finite strain consolidation computer models, cannot directly calculate consolidation of two slurried soils with different material properties, several runs were made to best simulate field behavior. First, a computer run was made to determine consolidation data for the contaminated dredged material and foundation soil, substituting a surcharge load for the 12.2-ft compressible cap; a separate run was then made to obtain consolidation information for the capping material. Results of these two simulations were superimposed to obtain the correct ultimate consolidation value for the mound and foundation, although the time-rate of consolidation is most likely too rapid since the contaminated material is assumed to be free-draining at its surface (in the first run), and water from this material is not considered in the consolidation calculations for the silt cap. A second analysis was made by assuming that both the contaminated dredged material and the silt cap had the properties of the former material. The third analysis was made assuming that the entire mound was composed of the silt capping material.

201. The predicted settlement for each of the three analyses is shown in Figure 44. The predicted settlement curves for the composite analysis and for the silt mound are very similar in both rate of consolidation and ultimate settlement; the ultimate settlement predicted was 12.5 ft for the composite analysis and 12.65 ft for the entirely silt mound. (Ultimate settlement is not shown in Figure 44.) The predicted settlement for 18.8 ft of contaminated dredged material is somewhat slower to occur, and the ultimate settlement for this mound was predicted to be 11.63 ft, which is approximately 1 ft less than the other predictions. Comparison of hydrographic survey data with predicted settlement curves for the STNH-S mound shows better agreement between the field data and the all-dredged material prediction during the early stages of

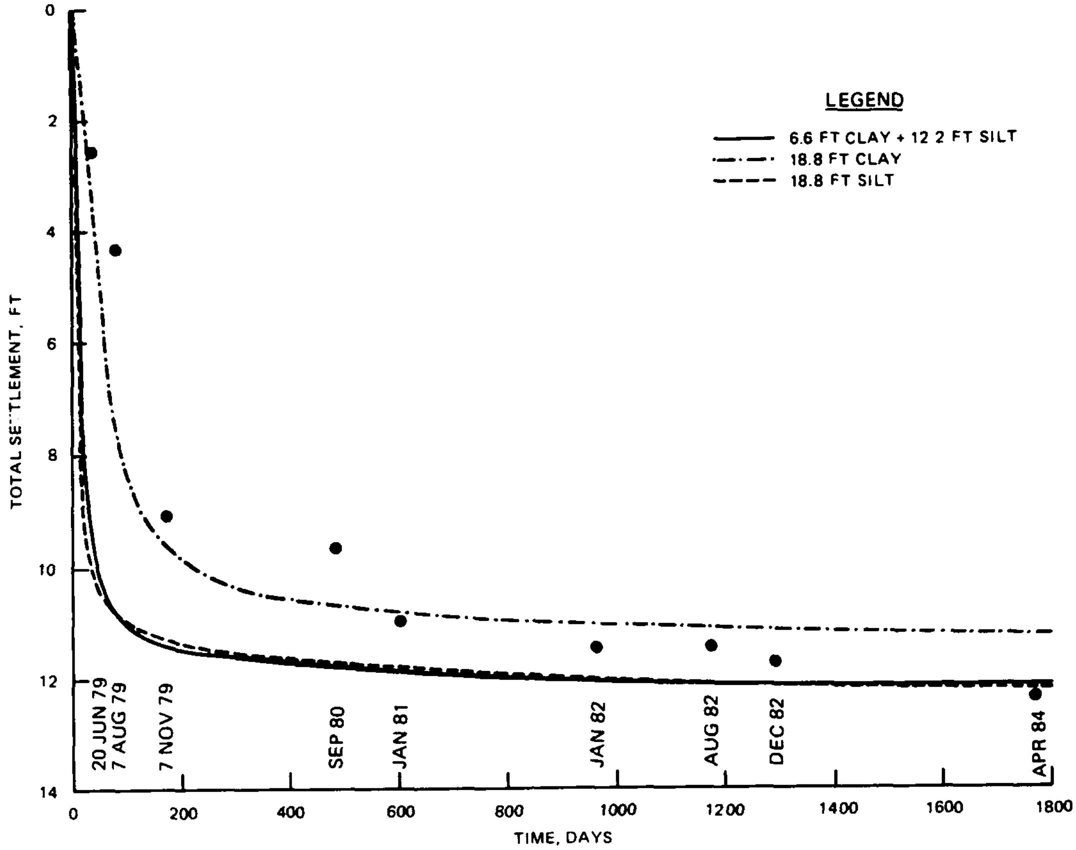

Figure 44. Predicted settlement for the three analyses of STNH-S mound

consolidation, while the field data trend downward toward the composite prediction during later stages of consolidation. This is the expected situation since the composite mound should correctly predict ultimate consolidation but should predict too rapid a rate of consolidation as a result of the modeling technique used. Despite the computer modeling limitations, very good agreement was obtained between the field data and the consolidation predictions. For purposes of comparison with settlement in other mounds, the settlement of the foundation, contaminated material, and compressible cap is shown separately in Figure 45 for the composite mound.

202. Because of the close agreement between the observed and predicted amounts and rates of consolidation for the STNH-S mound, it is felt that the passage of Hurricane David in 1979 had no significant effect on this mound, although such effect has previously been reported elsewhere (Morton 1980). Most likely, a large portion of initial consolidation occurred between the

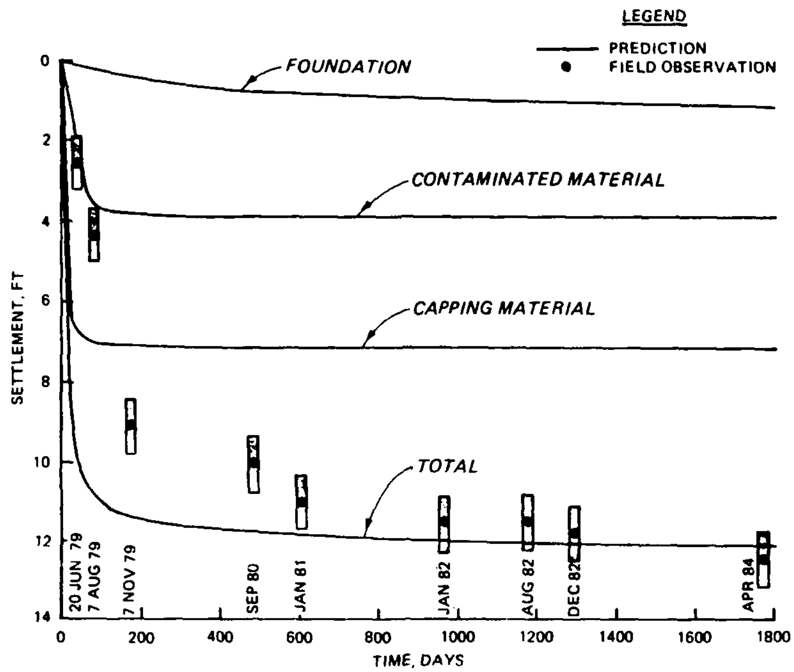

Figure 45. Time-rate of consolidation at center of the STNH-S mound

August 1979 and November 1979 monitoring activities, thus reducing the mound height from an initial 18.8 ft to approximately 10 ft by consolidation alone. Then, redistribution of surface material by Hurricane David, in conjunction with significant flattening of the mound resulting from consolidation, provided the impetus for interpretation of the changed mound geometry as a slope failure.

203. Predicted profiles of material characteristics within the 6.6-ft mound core are shown in Figures 46 and 47, respectively, for the composite analysis and for the entirely dredged material analysis. Comparison of the two figures indicates that the rate of consolidation predicted is more rapid in the composite analysis because the material is assumed to be free-draining at the surface of the 6.6-ft mound core. Since the mound as constructed has a fine-grained cap and thus the core is not free-draining, the profiles shown in Figure 47 are expected to be more representative of actual field conditions.

204. The predicted configuration of the STNH-S mound, after completion of 100-percent primary consolidation, is shown in Figure 48. The maximum height of the mound decreases from 18.8 ft to 6.3 ft above the original foundation elevation.

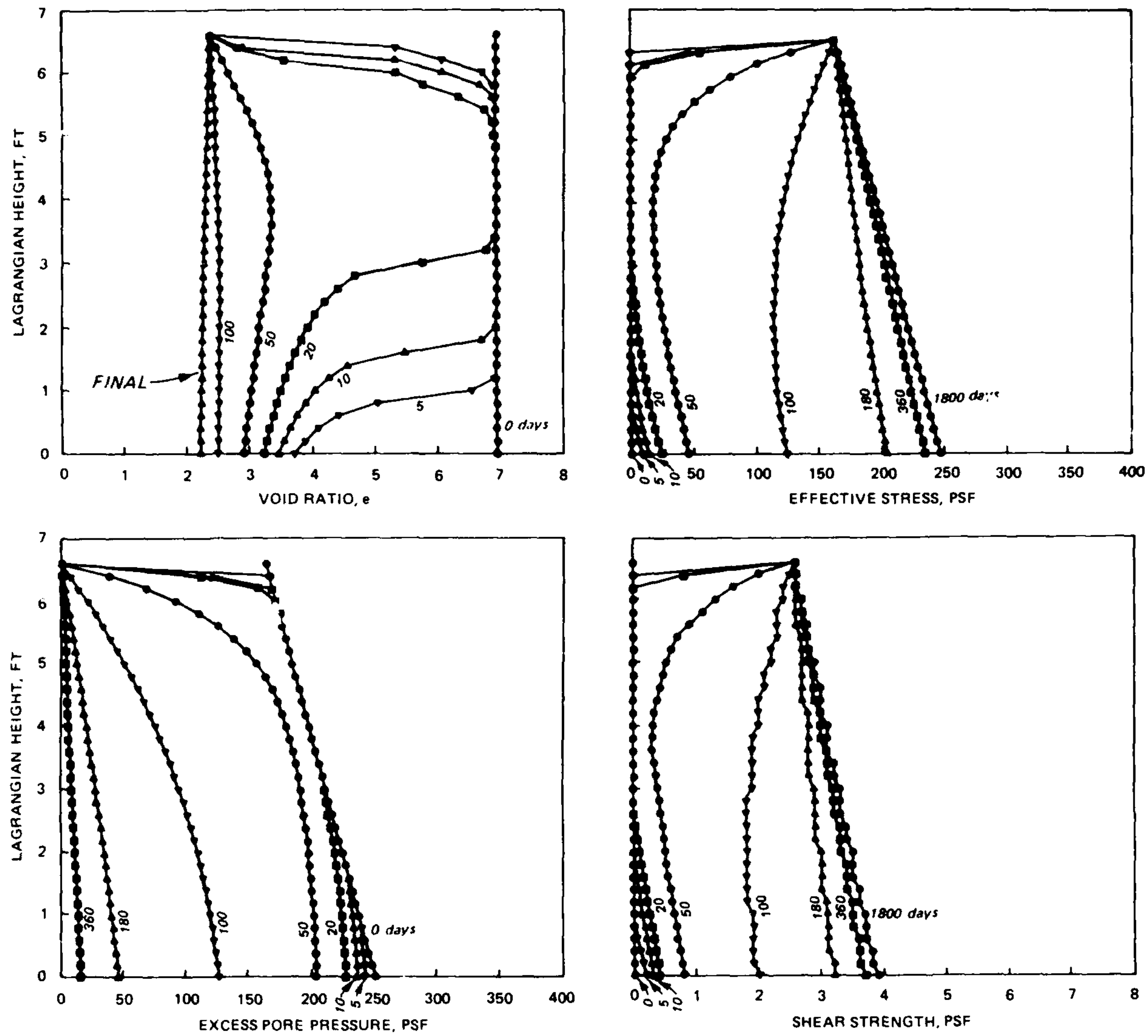

Figure 46. Temporal profiles of void ratio, effective stress, excess pore pressure, and shear strength for STNH-S mound (composite analysis)

205. At this site, the volume of the disposal site required to store the mounded dredged material is reduced by 66.5 percent, which corresponds to an increase in disposal site storage capacity of approximately 52,800 cu yd. This significant gain in remaining storage volume is the combined result of consolidation of the foundation soil, contaminated dredged material, and compressible capping material.

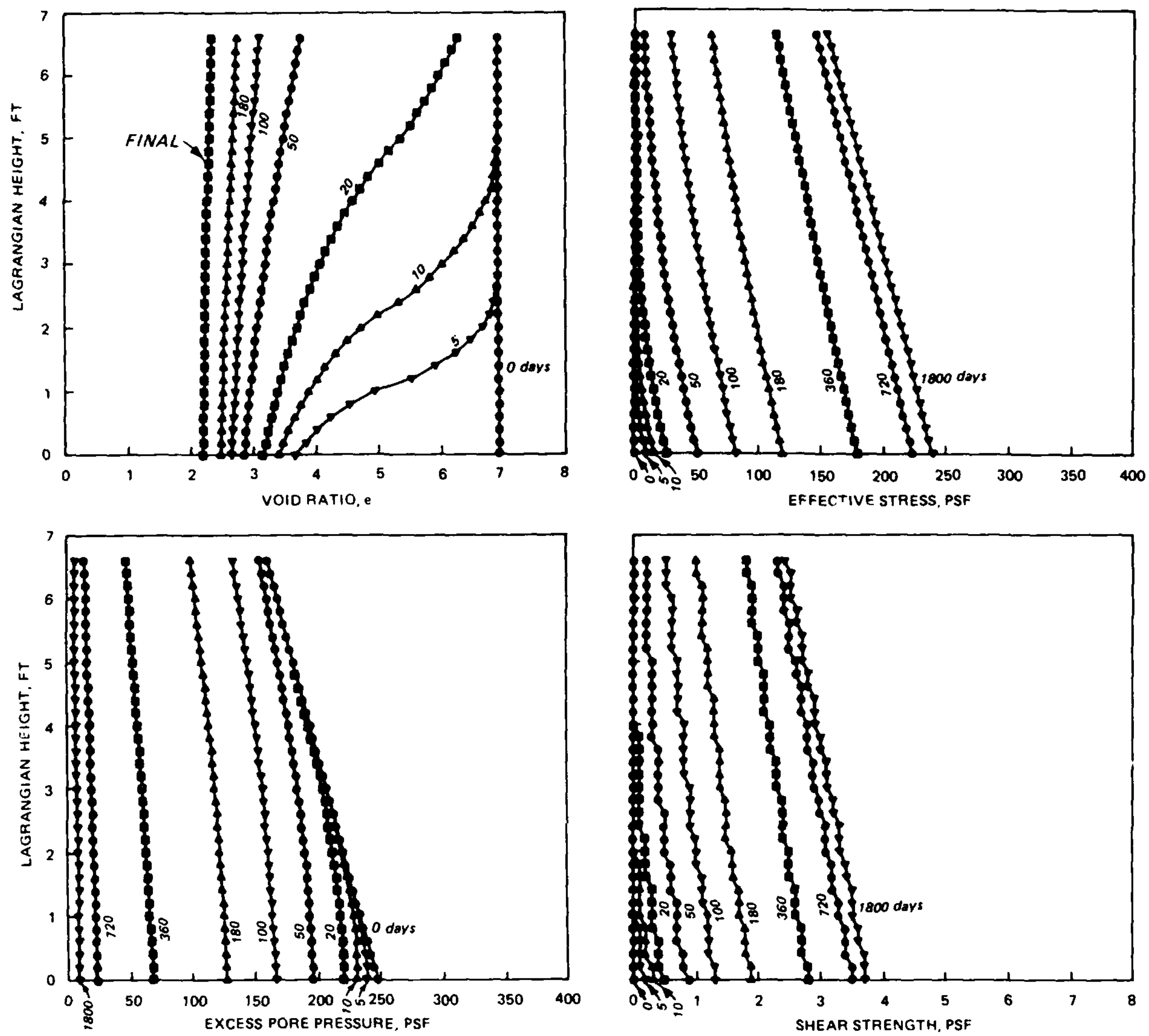

Figure 47. Temporal profiles of void ratio, effective stress, excess pore pressure, and shear strength for STNH-S mound (18.8 ft of dredged material analysis)

Comparison of Predictions

206. Although good agreement is obtained for each individual comparison of predicted and observed mound settlement, a trend seems to be apparent in all of the cases analyzed. In general, the field data tend to deviate from the predicted values by indicating that more ultimate settlement will occur than has been predicted. If this trend does exist and does not merely occur as the result of the hydrographic survey precision, then there are two plausible

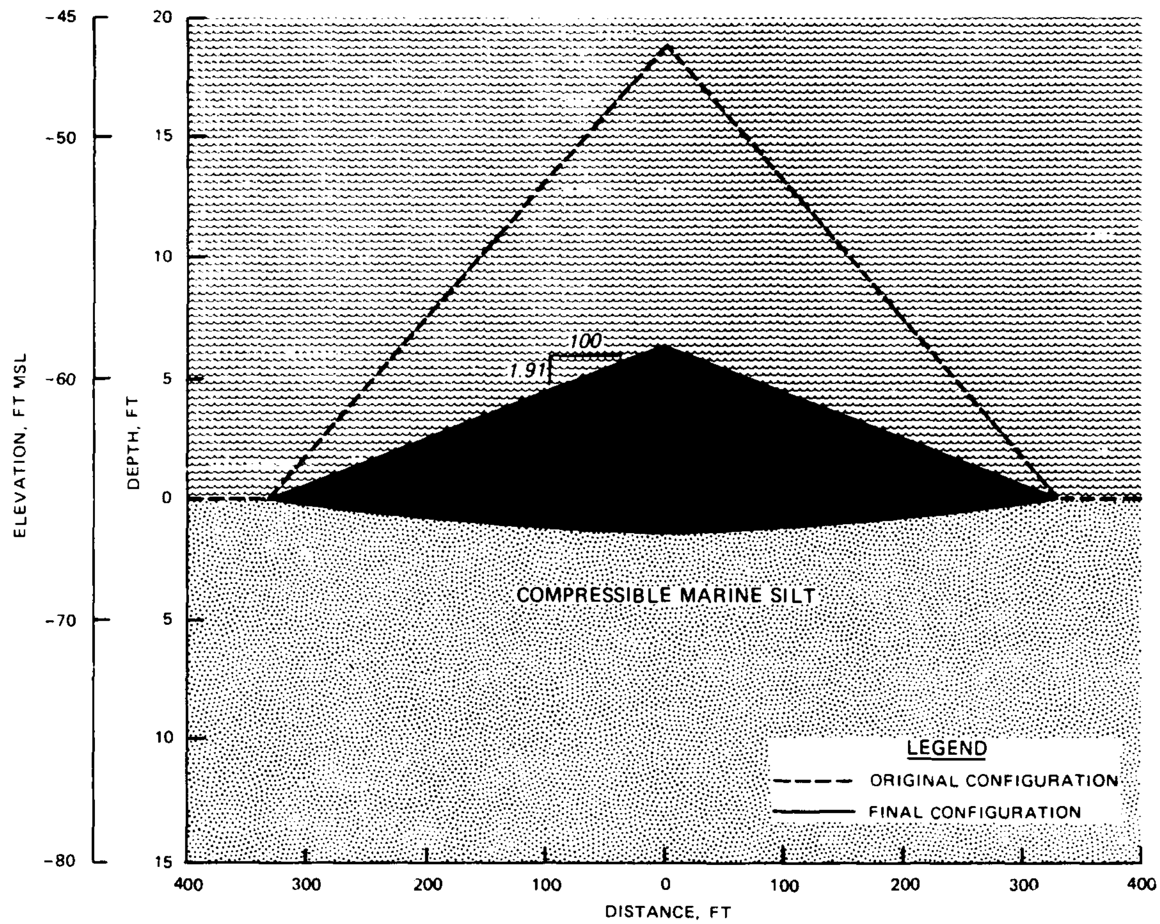

Figure 48. Predicted mound configuration at completion of consolidation of STNH-S mound

explanations. First, the mounds are three-dimensional, and a one-dimensional predictive technique has been used; therefore, multidimensional consolidation in the field may account for the difference between predictions and observations. Second, only primary consolidation has been predicted, while it is recognized that secondary compression can be significant in very soft soils such as dredged material; the occurrence of secondary compression would increase the amount of settlement observed.

207. Three of the mounds analyzed during this research (FVP, STNH-N, and STNH-S) were composed of compressible materials of similar geologic origin, were similar in height of contaminated material deposited, and had similar foundation soil conditions, although they represent three very different mound types. The FVP mound was composed entirely of the compressible material; it

was not capped. The STNH-N mound was capped with sand, while the STNH-S mound was capped with a compressible silt.

208. Comparison of the predicted settlement at each of these mounds shows the magnitude of variation in settlement that may be expected, depending upon mound design. The FVP mound, subjected only to self-weight consolidation, underwent approximately 1 ft of settlement (ignoring foundation settlement) whereas the compressible material in the STNH-N mound underwent much more settlement, about 4.3 ft, as a result of the free-draining sand cap. The STNH-S mound underwent about 3.9 ft of settlement within the contaminated material and 7.2 ft in the compressible silt cap, for a total settlement within the mound of 11.1 ft. Thus, the difference in settlement of mounded material can be expected to vary considerably depending not only upon the type of contaminated material in the mound (i.e., material compressibility and permeability) but also upon the design of the mound (i.e., loading and drainage conditions).

PART VIII: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

209. Current trends in dredged material disposal point toward more subaqueous (open-water) disposal of dredged material. Subaqueous placement of contaminated dredged material may require use of an uncontaminated (clean) capping material to isolate the contaminants from the overlying environment.

210. To determine the physical capacity of a subaqueous disposal site, the consolidation characteristics of the dredged material mound must be evaluated. As the use of subaqueous disposal sites increases, more emphasis must be placed upon determination of disposal site capacity.

211. When open-water disposal of dredged material results in formation of a mound of material with relatively flat slopes, the potential for consolidation can successfully be evaluated with one-dimensional consolidation theory.

212. Because of the large strains that often occur in soft dredged material, it is necessary to use the finite strain theory of consolidation to predict accurately the consolidation of subaqueous dredged material mounds. The governing equation of finite strain consolidation theory can be coded for computer solution; computer programs such as MOUND can be used to successfully predict consolidation of soft soil mounds.

213. Results obtained from the computer program MOUND provide information not only on the consolidation behavior of the mound, but also on the gains in shear strength which are to be expected as consolidation proceeds. The present predictive technique indicates that, although considerably greater increases in shear strength occur when a significant capping load is placed on the contaminated dredged material than when none is present, the soft dredged material will not develop shear strength comparable to typical soils. After field validation, shear strength information will also provide input or guidance for construction and use of subaqueous diked disposal areas.

214. The self-weight consolidation test and the standard oedometer test should be used to define the void ratio-effective stress and void ratio-permeability relationships needed for analysis of the postdisposal physical behavior of dredged material mounds. Use of the LSCRS test should be discontinued because of the expense and difficulty of conducting the test and the questionable nature of the data analysis procedures.

215. Although some improvements in the current testing methods and data analysis procedures will provide needed refinements, the laboratory methods and procedures presently in use provide consolidation characteristics for dredged material which can be used to accurately predict the performance of dredged material disposal areas.

216. Use of geologically or geotechnically trained personnel for fieldwork would facilitate collection of important data. Appropriately trained divers can provide proper interpretation of subaqueous disposal site conditions. Personnel experienced in geophysical investigation techniques can select appropriate methods and sounding frequencies to correctly identify subaqueous site characteristics.

217. The design of subaqueous mounds can significantly affect the amount of settlement to be expected in the mound material. Uncapped mounds of compressible material will undergo relatively little settlement in comparison to similar mounds with sand caps. Mounds with compressible caps will undergo significantly more settlement since both the contaminated material and the cap will consolidate.

218. The analysis presented in this work provides a systematic, organized approach for analyzing the behavior of subaqueous dredged material disposal sites. The methods include physical, chemical, and biological aspects, while placing major emphasis on the geotechnical engineering aspects of physical behavior.

Recommendations

219. Further investigations must be conducted to predict a priori the shape of the dredged material mounds formed when the material is placed in open-water disposal sites. Both the height and the diameter of the mounds must be predicted. This is essential for accurate site capacity determinations.

220. Previous work conducted by hydraulic engineers in studying erosion and resuspension has used a bed shear approach in which all soil particles were considered to be noncohesive. This approach must be modified to account for the cohesive nature of many of the dredged materials encountered in typical dredging projects. The increase in shear strength that occurs as consolidation proceeds should be considered along with the critical shear

velocities (bed shear stress) to predict the resuspension of cohesive particles (silt and clay).

221. Work should continue on development of a new constant rate of strain slurry consolidation test; this test must be evaluated after development is completed to determine whether it should replace the standard oedometer test for dredged material testing.

222. Since a major concern of placing any contaminated dredged material into subaqueous disposal sites is the potential release of contaminants into the water column, not simply the quantity of pore water extruded, the diffusion potential of the contaminants should be studied to determine the possible concentration and time-rate of release of various contaminants from the dredged material itself or from the mound's capping material. Both the initial concentration of various contaminants and the rate of diffusion must be considered in order to determine when chemical equilibrium will be reached.

223. After information is gained into the process of mound formation and the resulting mound shape, this process should be coded for computer analysis. This new code should be combined with the program MOUND and the best hydraulic model for resuspension/erosion to form a single, comprehensive computer model for analyzing subaqueous mound formation and behavior.

224. Although mound behavior can be analyzed for initial planning purposes by using a one-dimensional analysis, the two-dimensional effects involved in mound consolidation should be investigated. A two-dimensional approach should be developed for detailed site analysis. It will become more important to have an exact analysis procedure as these disposal sites are utilized subsequently for disposal operations over a number of years.

225. Although the procedure outlined in this report for analysis of subaqueous dredged material disposal mounds will allow a determination of site capacity to be made, the accuracy of the computer model must continue to be verified. Field investigations should be conducted to monitor the consolidation behavior of dredged material mounds over time. At the same time, samples of the dredged material should be collected and subjected to laboratory testing so that analyses can be made to predict the mound behavior. These predictions should then be compared with the field monitoring data to determine the validity of the predictions and/or identify needed modifications to the predictive model.

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APPENDIX A: CLASSIFICATION TEST RESULTS

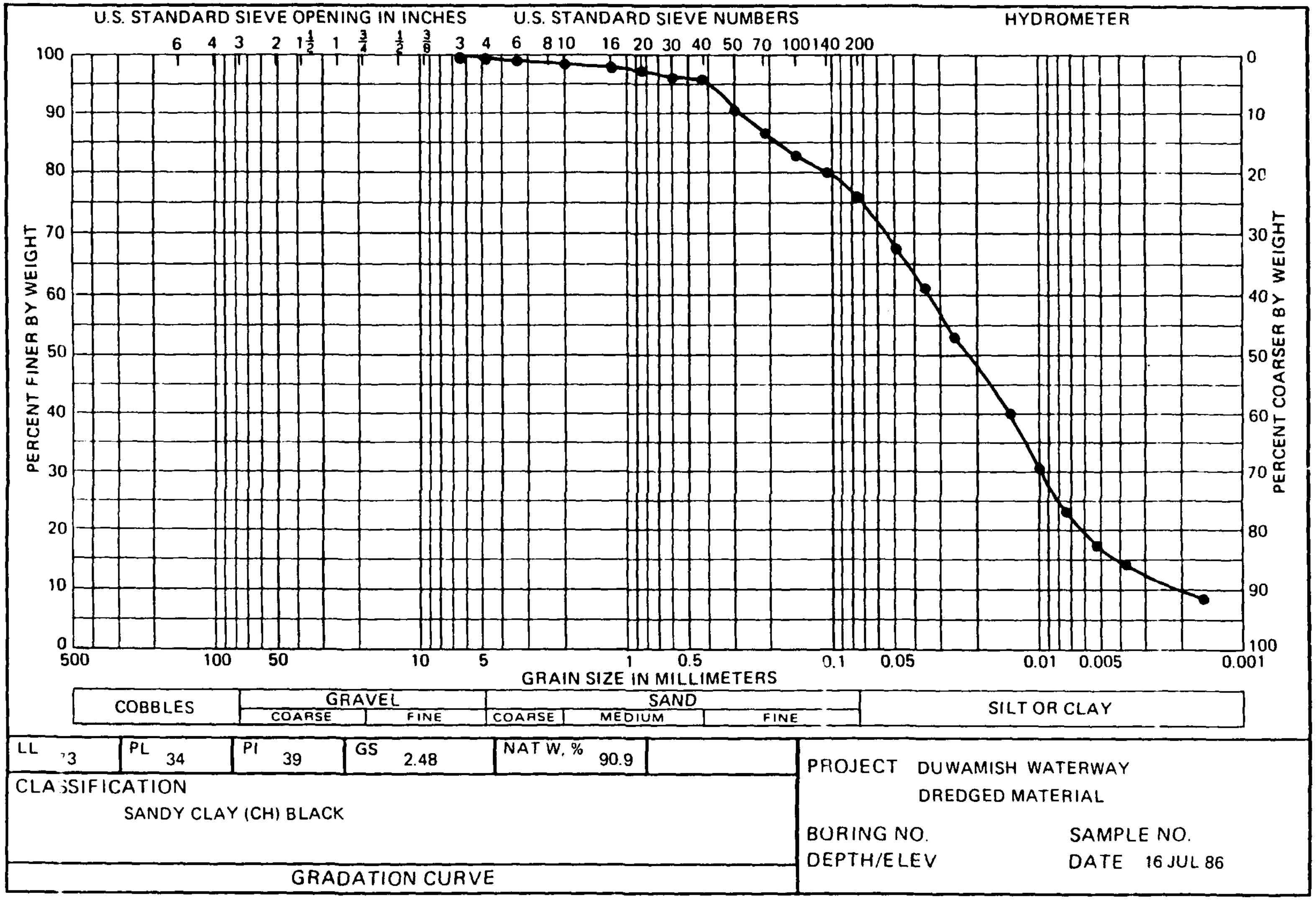
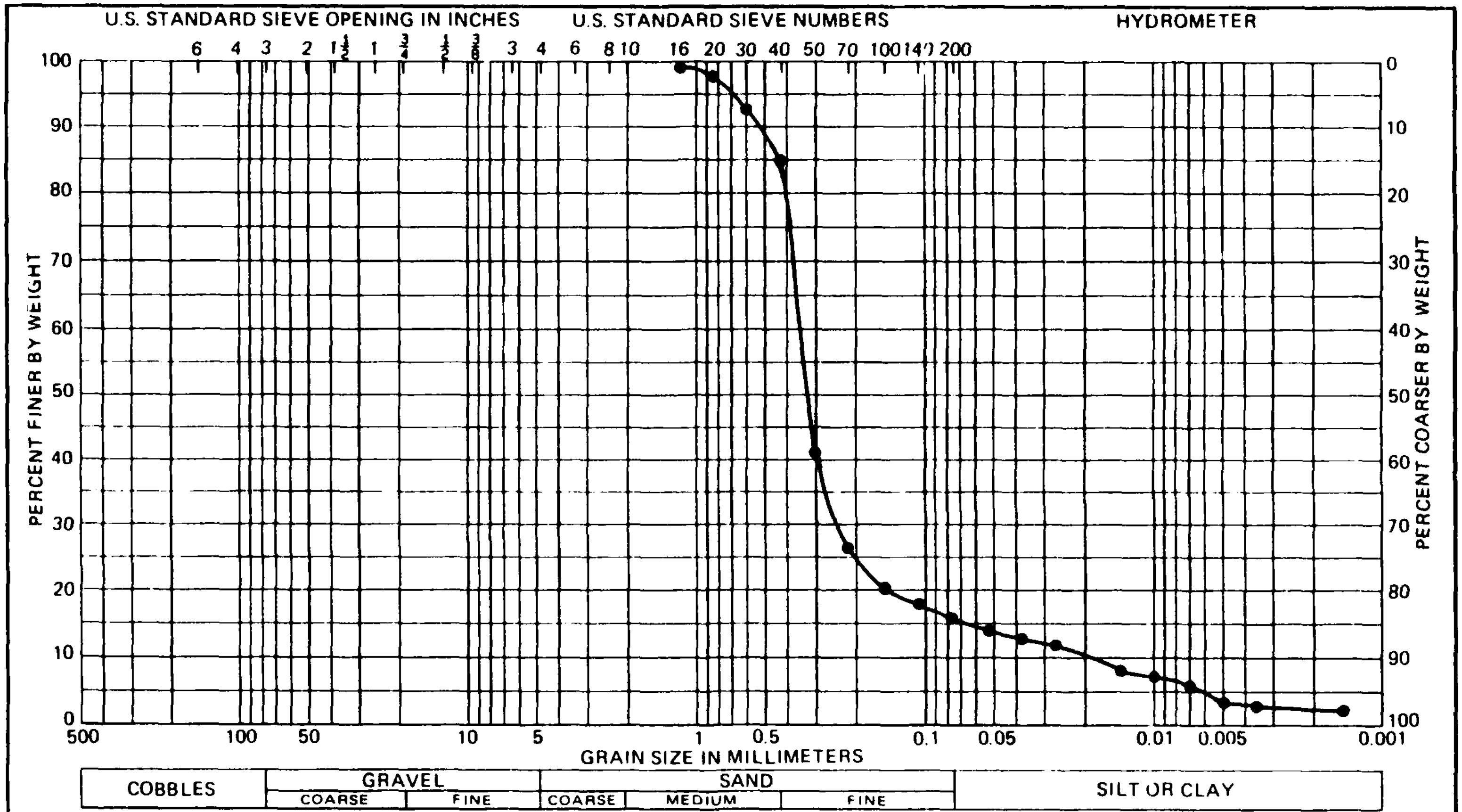


Figure A1. Duwamish Waterway dredged material



LL NP	PL	PI	GS 2.78	NAT W, % 61.7	

CLASSIFICATION
SILTY SAND (SM) DARK GRAY

GRADATION CURVE

PROJECT DUWAMISH WATERWAY CAP

BORING NO. COMPOSITE SAMPLE NO.
DEPTH/ELEV DATE 16 JUL 86

Figure A2. Duwamish Waterway capping material

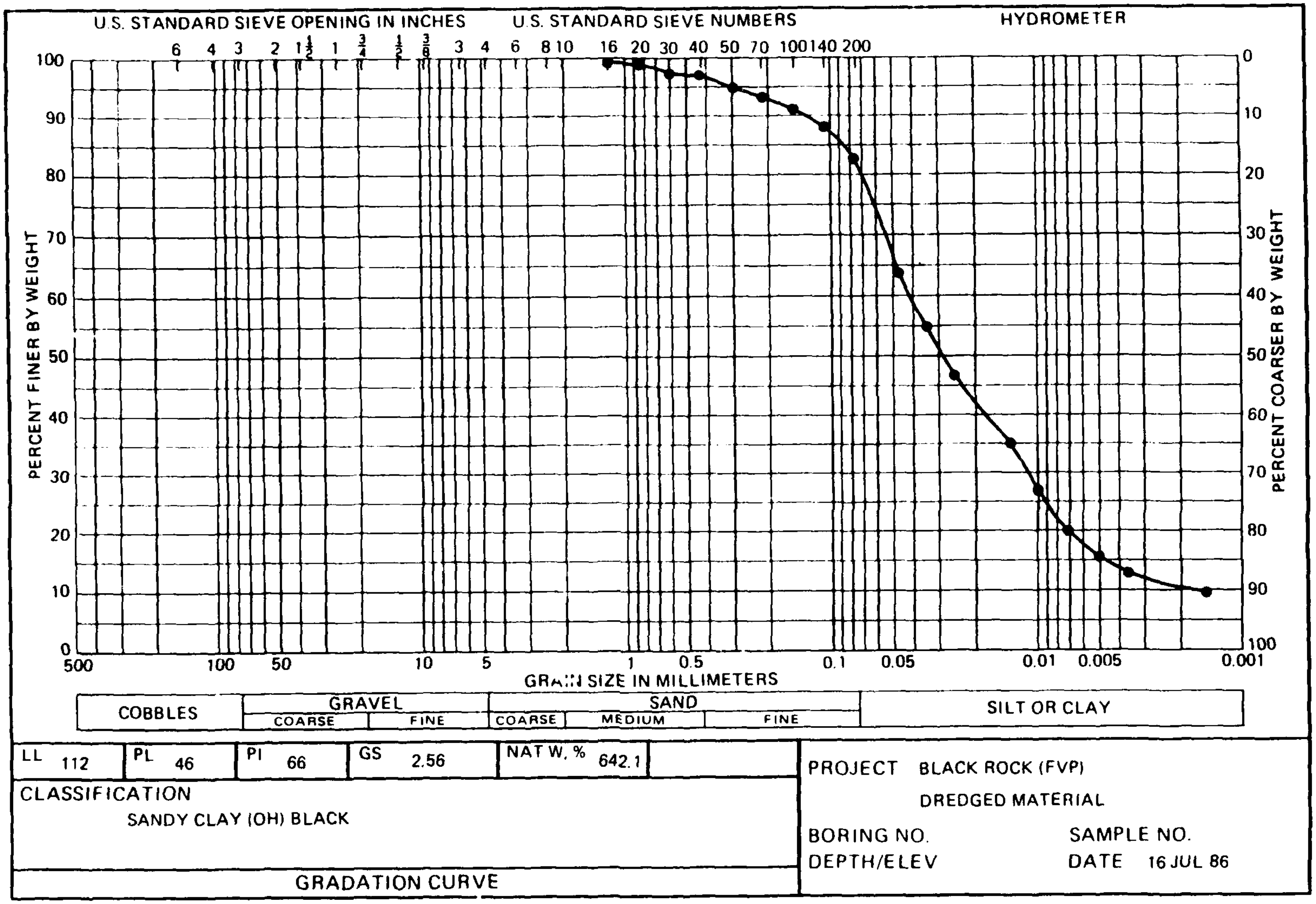
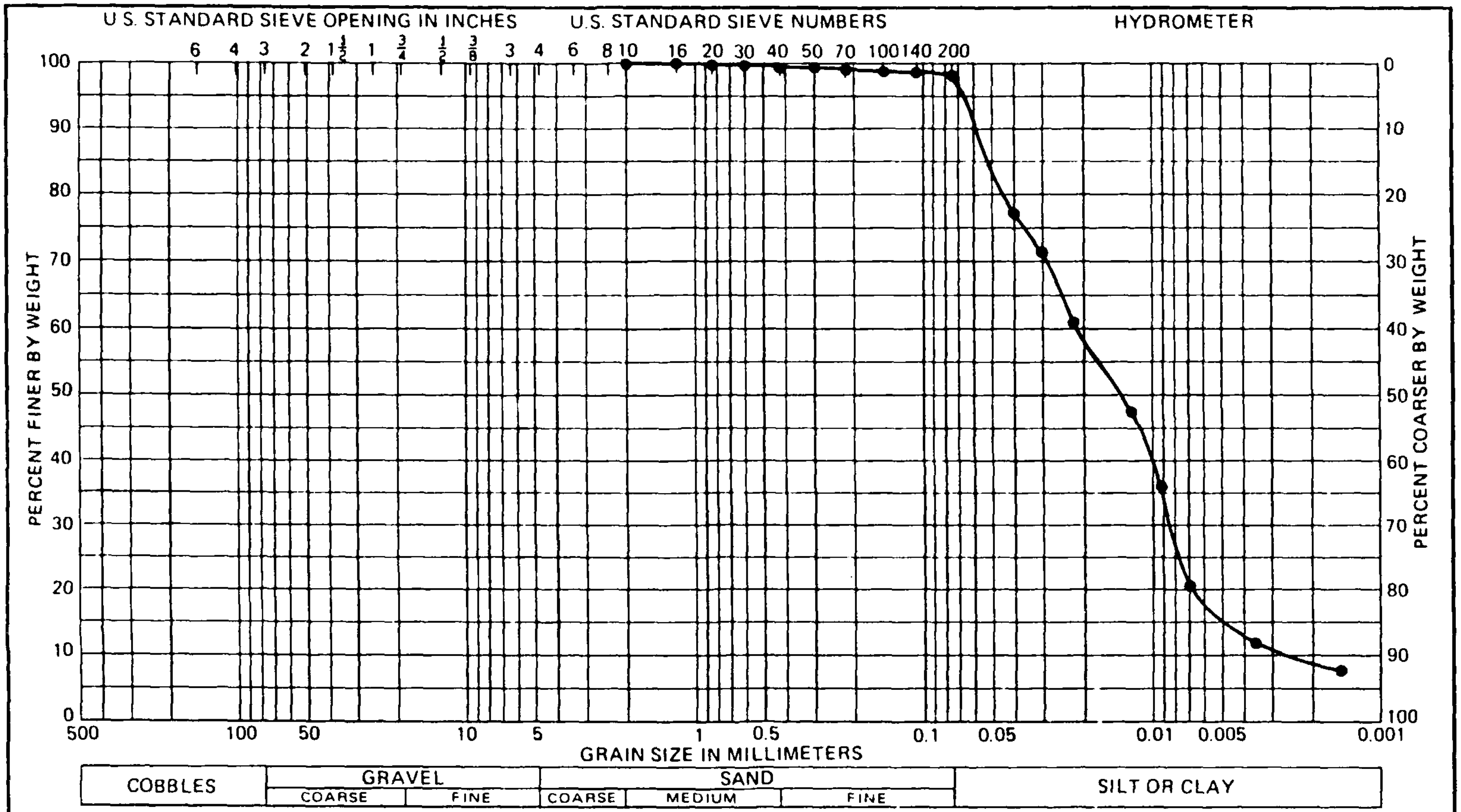


Figure A3. Field Verification Program dredged material



LL 69	PL 34	PI 35	GS 2.67	NAT W, % 100.6	

CLASSIFICATION
SILT (MH) GRAY

GRADATION CURVE

PROJECT CENTRAL LONG ISLAND SOUND FOUNDATION SOIL

BORING NO.
SAMPLE NO. 1
DEPTH/ELEV 4'
DATE 18 AUG 87

Figure A4. Central Long Island Sound foundation soil

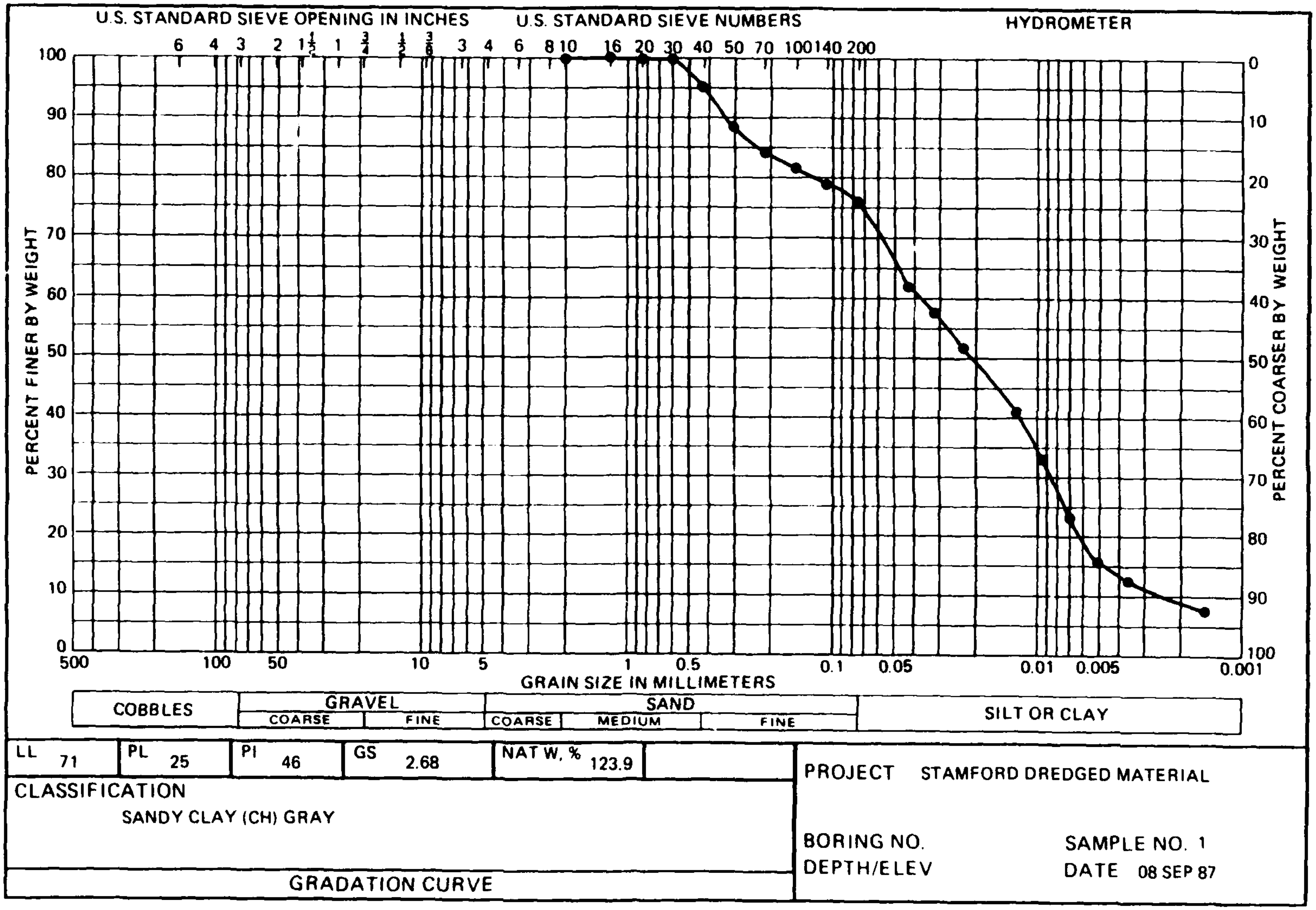


Figure A5. Stamford dredged material

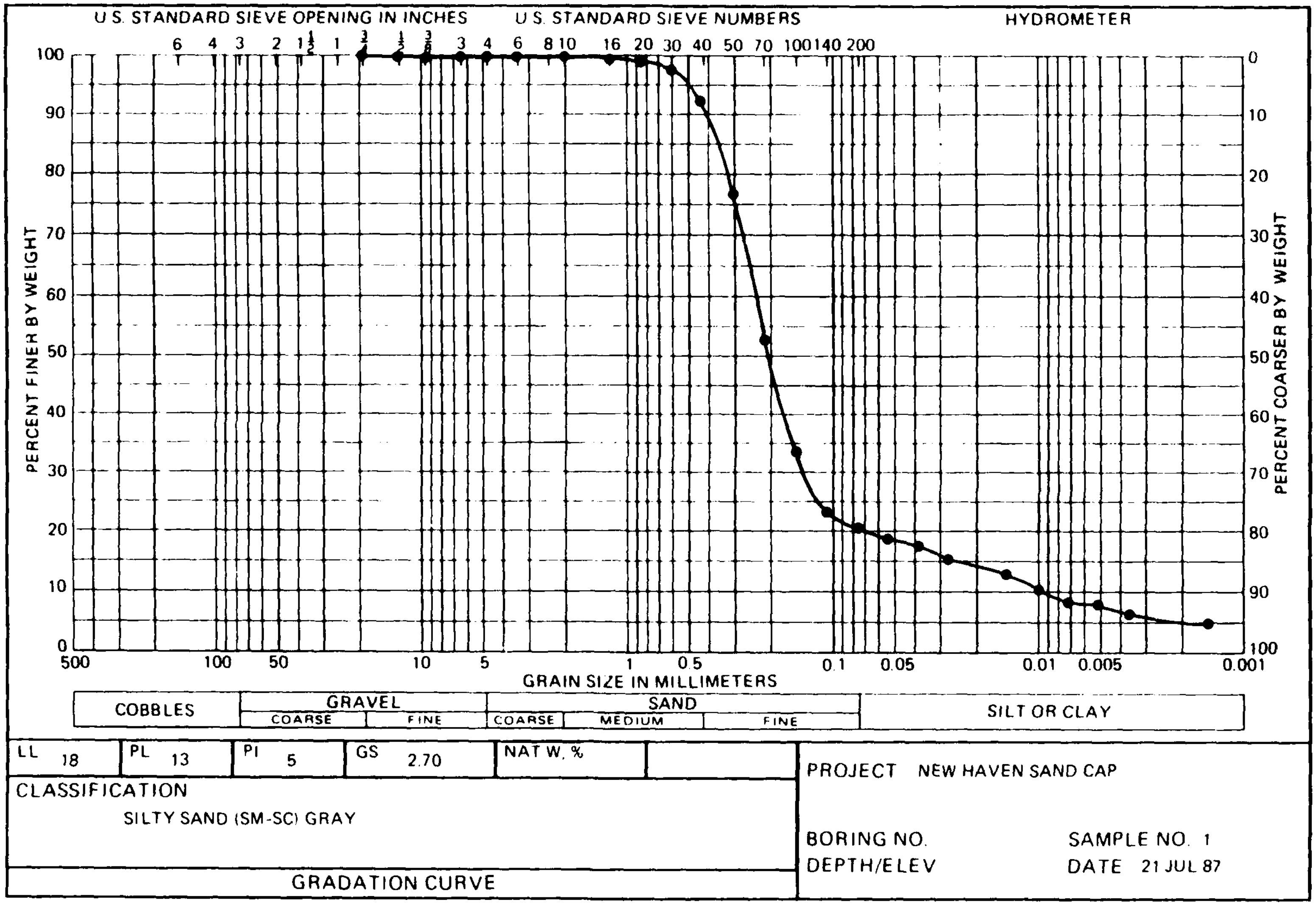
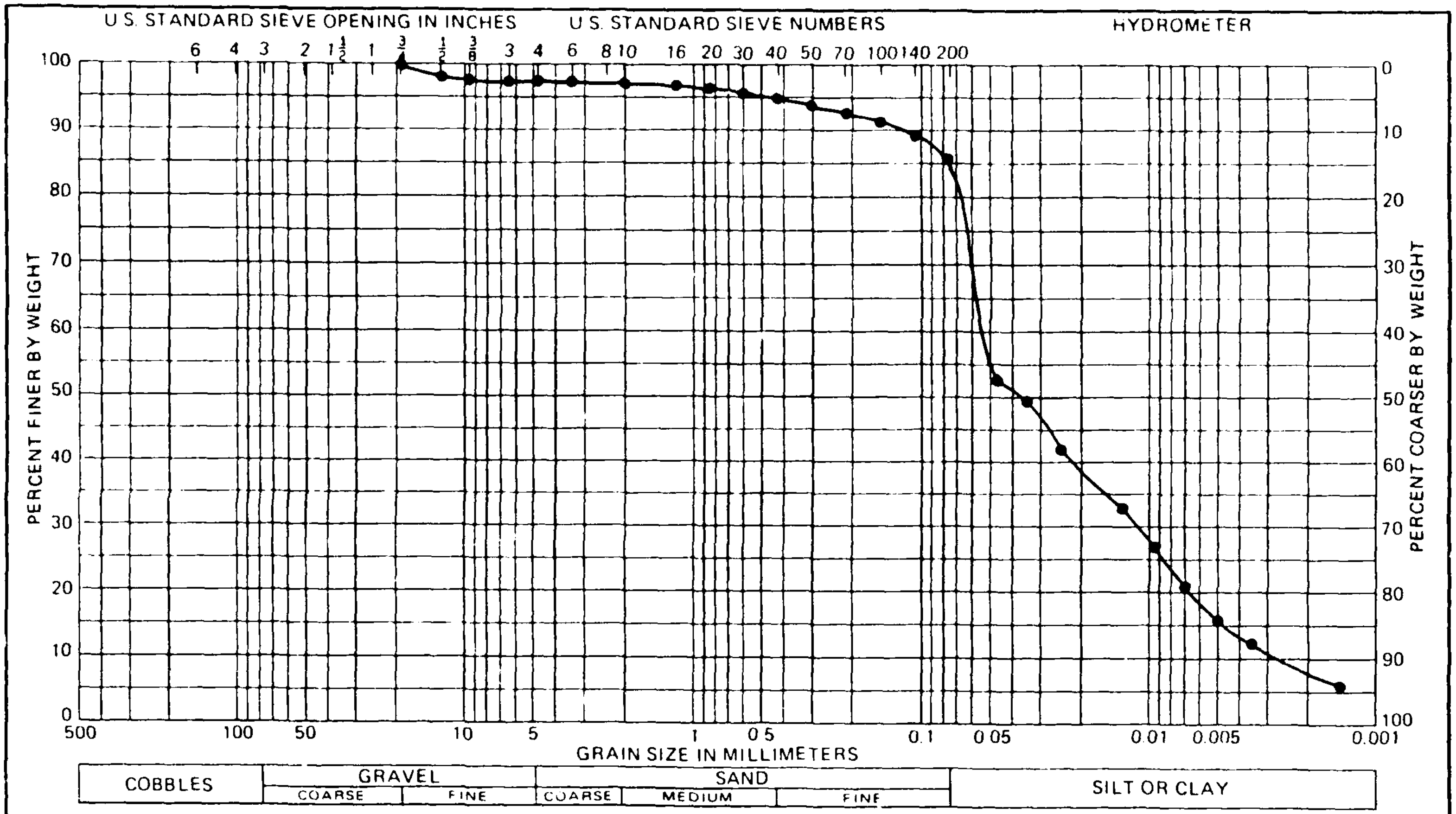


Figure A6. New Haven sand capping material



LL 96	PL 28	PI 68	GS 2.69	NAT W. % 132.5

CLASSIFICATION

CLAY (CH) GRAY; WITH SAND

INSUFFICIENT SAMPLE FOR ACCURATE GRADATION

GRADATION CURVE

PROJECT NEW HAVEN SILT CAP

BORING NO.

SAMPLE NO. 1

DEPTH/ELEV

DATE 31 JUL 87

Figure A7. New Haven silt capping material